Features of Various Offshore Structures

by
Joseph Peraino, Burr L. Chase,
Tomasz Plodowski, and Lydon Amy

MISCELLANEOUS PAPER NO. 3-75
APRIL 1975

Prepared for
U. S. ARMY, CORPS OF ENGINEERS
COASTAL ENGINEERING
RESEARCH CENTER
Kingman Building
Fort Belvoir Va. 22060

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REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: MP 3-75	2. GOVT ACCESSION NO.
3. RECIPIENT'S CATALOG NUMBER	
4. TITLE (and Subtitle): FEATURES OF VARIOUS OFFSHORE STRUCTURES	5. TYPE OF REPORT & PERIOD COVERED: Miscellaneous Paper
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): Joseph Peraino, Burr L. Chase, Tomasz Plodowski, and Lydon Amy	8. CONTRACT OR GRANT NUMBER(s): DACW 72-73-C-0011
9. PERFORMING ORGANIZATION NAME AND ADDRESS: Raymond Technical Facilities, Incorporated, Two Pennsylvania Plaza, New York, NY 10001	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS: B31234
11. CONTROLLING OFFICE NAME AND ADDRESS: Department of the Army, Coastal Engineering Research Center (CEREN-DE), Kingman Building, Fort Belvoir, Virginia 22060	12. REPORT DATE: April 1975
	13. NUMBER OF PAGES: 113
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)	15. SECURITY CLASS. (of this report): UNCLASSIFIED
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Breakwaters
Coastal Engineering
Offshore Structures
Oil Drilling Islands
Oil Drilling Platforms

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The growing shortage of suitable waterfront sites for industrial complexes, transportation facilities, marine terminals, and recreation, and the increasing concern to preserve our natural coastal environment, and the continuing need to obtain the advantages of such locations has forced a consideration of artificial means to satisfy this need. In addition, the economics of larger-capacity, deeper-draft vessels have outmoded our present U.S. harbors and possible coastal sites for these newer ships.

This report provides a means of comparison for various offshore structures from the technical, environmental, and economic aspects through the classification and identification of some existing structures.

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PREFACE

This report is published to provide coastal engineers, through the classification and identification of some existing offshore structures, a means of comparison for the various structures from the technical, environmental, and economical aspects in investigating the feasibility of designing and constructing deepwater ports to accommodate deep-draft ocean vessels. The work was carried out under the coastal construction research program of the U.S. Army Coastal Engineering Research Center (CERC).

The report was prepared by Joseph Peraino, Burr L. Chase, Tomasz Plodowski, and Lydon Amy, under CERC Contract No. DACW72-73-C-0011 with Raymond Technical Facilities, Inc.

The authors gratefully acknowledge the generous assistance and encouragement provided by CERC personnel during preparation of this study. Also invaluable were the cooperation and assistance furnished by the U.S. Army Engineer Districts, and the owners, engineers and contractors who were contacted on many details of the study.

Engineering data for a few offshore structures are incomplete and were unobtainable from the sources contacted. It is hoped that users of this report can expand or update its content by identifying or submitting data sheets on these or other offshore structures.

Dr. J. Richard Weggel was the CERC contract monitor for the report, under the general supervision of Mr. Robert A. Jachowski, Chief, Design Branch, Engineering Development Division.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

JAMES L. TRAYERS
Colonel, Corps of Engineers
Commander and Director

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FEATURES OF VARIOUS OFFSHORE STRUCTURES

by

Joseph Peraino, Burr L. Chase,
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I. INTRODUCTION

The growing shortage of suitable waterfront sites for industrial complexes, transportation facilities, marine terminals, recreational use, and of the increasing concern to preserve what remains of our natural coastal environment, and the continuing need to obtain the advantages of such locations are forcing people to turn to artificial means to satisfy this need. Compounding these difficulties, the economics of larger capacity, deeper-draft vessels have brought about the outmoding of our present harbors and possible coastal sites for these newer ships.

Until the advent of these large deep-draft carriers, our natural harbors along the coasts of the continental United States, well protected from the open sea, and well developed for the trade involved, have answered our needs satisfactorily. Furthermore, the availability of such harbors plus the general exposure to long fetches of ocean along both coasts have until now discouraged any efforts toward moving offshore in order to reach berthing space with the necessary depth requirements. However, now, under the changed conditions of ocean shipping, the feasibility of such a move must be reconsidered.

Through the classification and identification of certain existing offshore structures, this interim study is an attempt to provide a means of comparison for the various structures and types of structures from the technical, environmental, and economical aspects.

For the purposes of this study, an offshore structure is any permanent, fixed structure in an ocean or estuarine location, essentially unconnected to shore. A physical tie which does not influence the effect of the structure on its environment will not be considered a shore connection.

II. GRAVITY STRUCTURES

1. Chesapeake Bay Bridge Tunnel, Portal Islands, Cape Henry—Cape Charles, Virginia.

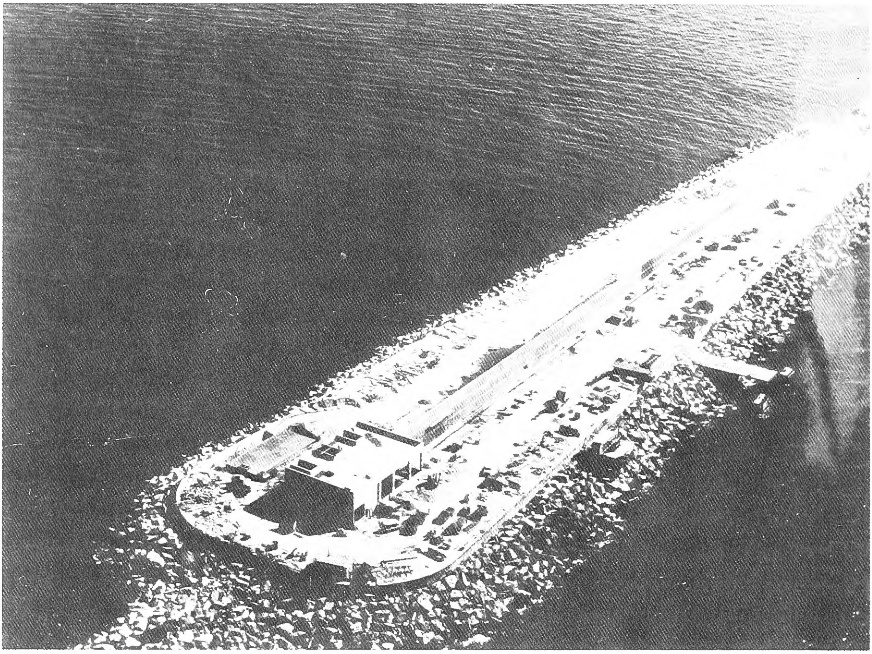

Figure 1. Aerial oblique view of Portal Island No. 1 from northwest.

a. Construction.

(1) Completed. April 1964.

(2) Type. Fill-type artificial islands protected by riprap armor stone weighing up to 20 tons (Fig. 1).

(3) Purpose. Transition structures between bridge trestle and tunnels.

b. Owner. Chesapeake Bay Bridge and Tunnel District, P.O. Box 111, Cape Charles, Virginia 23310.

c. Location (approximate). Mouth of Chesapeake Bay.

Latitude: 36°00′N. – Longitude: 76°06′W.

d. Physical Environment (Fig. 2).

(1) Protected Ocean-Estuarine Environment. The islands are protected from the northeast and south by the mainland and exposed to the Atlantic Ocean to the east. Exposed to waves generated within Chesapeake Bay by northerly winds.

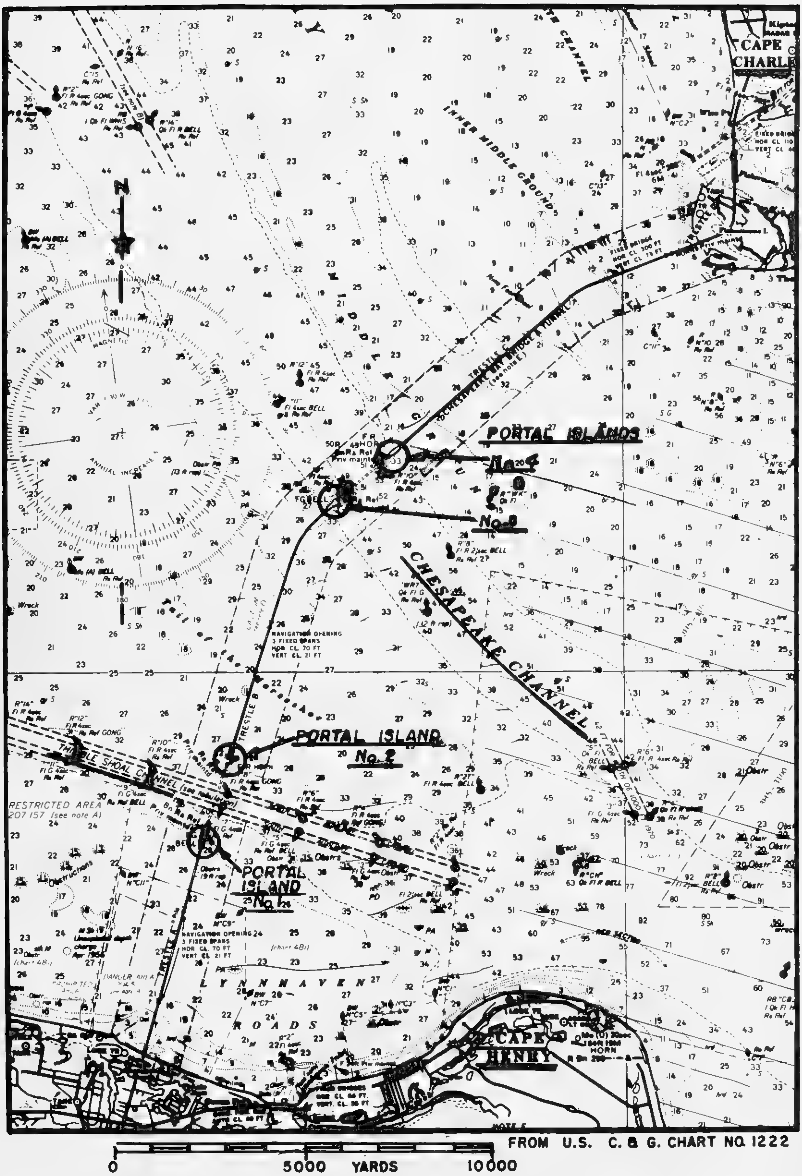

Figure 2. Location plan.

(2) Wave Conditions. Due to ocean exposure to the east and bay exposure to the north, maximum wave heights during the winter are normally 8 to 10 feet.

(3) Currents. Basically tidal currents, ranging up to an approximate maximum speed of 2 knots.

(4) Winds. Prevailing winds are from the southwest with an average measured speed of 11 knots, and a maximum of about 70 knots (U.S. Coast and Geodetic Survey, 1966).

(5) Storm Surge and Tides (U.S. Army Engineer District, Norfolk, 1973).

Mean tidal range, 2.5 feet

Mean spring tidal range, 3.0 feet

Extreme high tides, 7.0 feet

Extreme low tides, –3.0 feet

(6) Sediment Conditions. Medium compact to compact fine sand, with some silt; trace of shells (Sverdrup and Parcel, 1961).

e. Structural Features (Sverdrup and Parcel, 1961) (Figs. 3 and 4).

(1) Dimensions of Basic Structure.

MLW (approximate):

Length, 1,500 feet

Average width, 220 feet

Area, 8 acres

Finished grade (approximate): 5 acres

(2) Side Slopes. 2 on 1.

(3) Finished Grade. 30 feet above MLW.

f. Design Data. Preliminary design provided for possibility of two layers of protective armor, but outer layer was finally considered unnecessary.

(1) Design Conditions (Beach Erosion Board, 1960).

Depth at structure, 32.0 feet at MLW

Astronomical tide, 2.5 feet at MLW

Storm surge, 10.0 feet at MLW

Maximum water level, 12.5 feet above MLW

Design wave. Significant height, 17.5 feet; period, 8.9 seconds; windspeed, 105 mph hurricane (100-year storm)

Heavy riprap (Type A), 10 tons per unit; minimum size specified (±20 tons used as it came from quarry)

(2) Model Study (Beach Erosion Board, 1960). A model study was performed at the Beach Erosion Board to confirm the results of design computations with stone sizes versus wave conditions. Five tests were run with varying combinations of wave height, water depth, and number of layers of cover stone. The designed riprap used for cover stone was distributed as follows: Light stone, 6 to 8, and 8 to 10 tons per unit (5 percent each), and heavy stone, 10 to 11 tons per unit (90 percent).

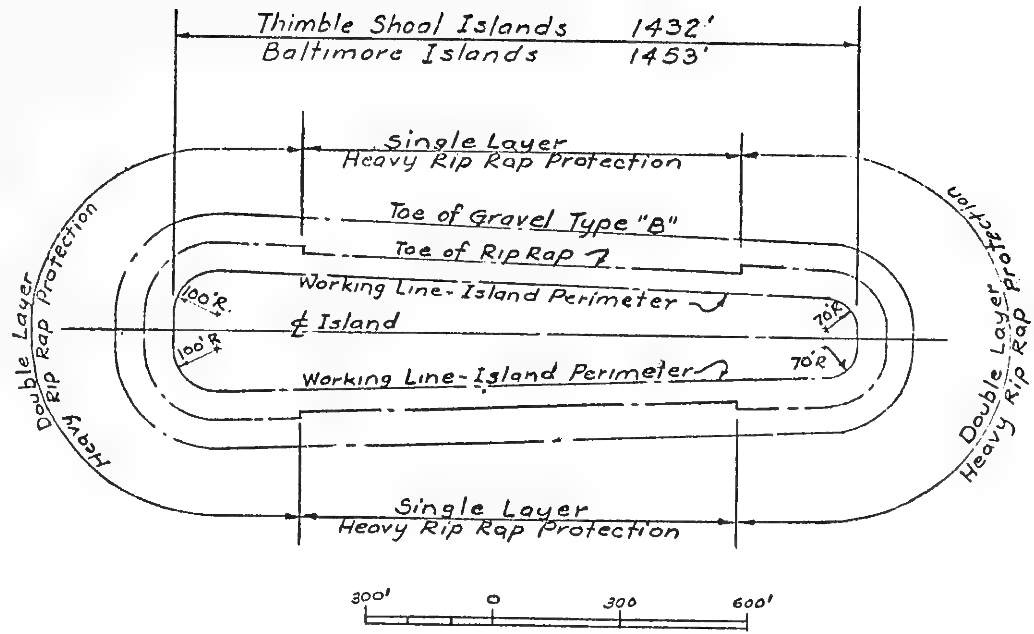

Figure 3. Typical plan of islands (Sverdrup and Parcel, 1961).

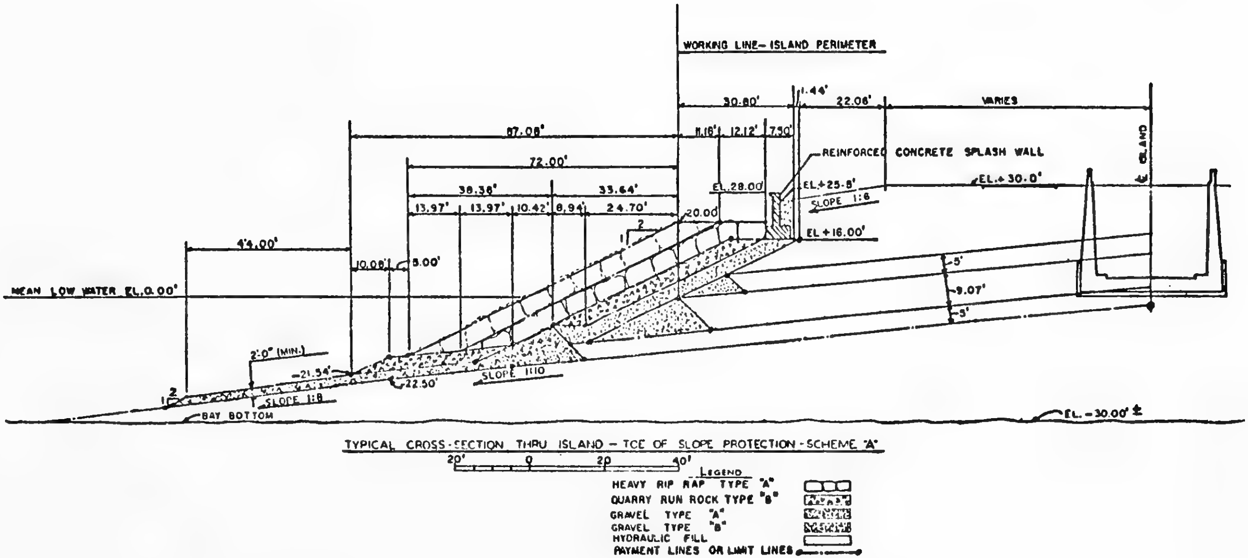

Figure 4. Typical cross section through island (toe of slope protection).

With a significant wave 14 feet high (expected to occur every 2 years) some overtopping occurred in the model. During these tests, gravel at the toe underlying the rock was in continuous motion.

With a significant wave 17.5 feet high, the entire slope slumped after the first few waves. There was a high degree of overtopping and considerable force and pressure against the seawall; cover stone was removed, exposing gravel and undermining the wall.

Due to infrequency of a 17.5-foot wave, design size of protective riprap was not changed. Because of salinity of water at site, it was recommended that stone size be increased by 10 percent.

(3) Instrumentation and Observation (U.S. Army Engineer District, Norfolk, 1971). A wave gage has been installed on the fishing pier off Portal Island No. 1 by CERC (Fig. 1).

Settlement plates are distributed over the finished grade area of the islands (and within the tunnels) for checking settlement by the Bridge and Tunnel District.

Rows of pins are set in the armor stone, transversely across the islands, Nos. 1 and 3, for checking horizontal and vertical movement of the armor stone. This is carried out by means of surface surveys by the U.S. Army Engineer District, Norfolk, and checked by aerial surveys. Underwater surveys are accomplished for checking armor stone movement and changes in bottom topography near the structures.

Extensive hydrographic surveys are also performed over the entire project by the Bridge and Tunnel District.

g. *Structural Performance* (U.S. Army Engineer District, Norfolk, 1971; Sverdrup and Parcel, 1972). Structures are performing satisfactorily, except that one island has experienced some settlement (up to 2 feet at one end) due to localized foundation conditions. Design wave heights and winds have not been experienced to date. Structures have experienced no damage from environmental conditions; hence, no repair work has been necessary. Maintenance has not been performed and none is planned.

Regular surveillance has been performed on the armor stone. The results indicate little or no horizontal or vertical movement. Aerial photos, checked by surface survey, cover above-water areas, and hydrographic surveys cover underwater areas.

The finished grade of all islands was surfaced with asphalt to prevent wind erosion of sandfill shortly after construction. Routine maintenance of the surface is required at several locations.

Splash walls are still in good condition except for an isolated area (Sverdrup and Parcel, 1972). Stabilization of the wall at this point is being considered.

There have been several incidents of ships or barges colliding with the trestle part of the project, involving repair work consisting of new piles and replacement sections, ordinarily held in stock. There are no records of the islands being hit.

h. Effect of Structures on Environment.

(1) Physical. The bottom topography in the vicinity of the islands has changed between the regular hydrographic surveys by the Corps of Engineers with a general pattern

developing offshore of the revetments. Slight scour is evident at the toe of the armor stone some distance around each island with slight accretion of sand away from the islands (U.S. Army Engineer District, Norfolk, 1971).

In addition, scour has been noted along the line of the trestle extending from each island (Sverdrup and Parcel, 1972).

The extent of scour both in area and depth has varied at each island, but has generally been centered on the fifth or sixth trestle bent off the island. A close check has been kept on the extent of scour at each island. The scour to the north of the northernmost island has been by far the most severe of the four areas and remedial action has been taken here in the form of dikes and riprap.

Figure 5 shows scour conditions and a comparison of recent soundings with original bottom contours at Island No. 4. For further comparison, the approximate depth of scour at each island, according to similar records is as follows: Island Nos. 1, 8 feet; 2, 8 feet; 3, 11 feet; and 4, 20 feet.

Sport fishing and small boat activity, the crossing in general and the islands in particular, have contributed greatly to the recreational activity in the area. The biota is discussed in the following paragraph. The psychological effect of having the structures available for emergency protection has increased small boat activity across the mouth of Chesapeake Bay.

(2) Biota (U.S. Army Engineer District, Norfolk, 1973). The islands, constructed with open rock faces, provide a habitat for numerous types of sea plants and animals. The naturally protected areas between the rocks attract large quantities of small fish, shellfish and crabs. These small fish in turn provide a food supply for the larger species which gather at the islands in great numbers.

Lacking a specific count of fish in the area before construction makes it impossible to determine quantitatively the increase in fish population. However, in the opinion of local sport fishermen, the number and variety of fish have greatly increased.

Construction of a fishing pier off Island No. 1, provides an opportunity for local fishermen and travelers to take advantage of these conditions.

Wildlife presently in the area are listed in Tables 1 and 2 (U.S. Army Engineer District, Norfolk, 1973).

(3) Aesthetics. While obviously affecting the view across the mouth of Chesapeake Bay, the need for the crossing and the effort expended on its overall appearance have resulted in general acceptance including ASCE designation as "The Outstanding Engineering Achievement—1965."

i. Engineering. Sverdrup and Parcel, Consulting Engineers, 915 Olive Street, St. Louis, Missouri 63101.

j. Construction Contractors (joint venture). Tidewater Construction Corporation, P.O. Box 57, Norfolk, Virginia 23601 (Sponsor); Merritt-Chapman and Scott Corporation, New York, New York 10001 (discontinued operations); Raymond International Incorporated,

2801 South Post Oak Road, Houston, Texas 77027; and Peter Kiewit Sons' Company, 1000 Kiewit Plaza, Omaha, Nebraska 68131.

k. Construction Dates. October 1960 to April 1964.

l. Construction Cost.

(1) Contract Value. $139,200,000 (total project including construction of islands, bridges, and tunnels).

(2) Fill for the Four Islands of the Project (approximate): Hydraulic fill, 4,000,000 cubic yards; stone fill, 870,000 tons; and heavy riprap, 300,000 tons.

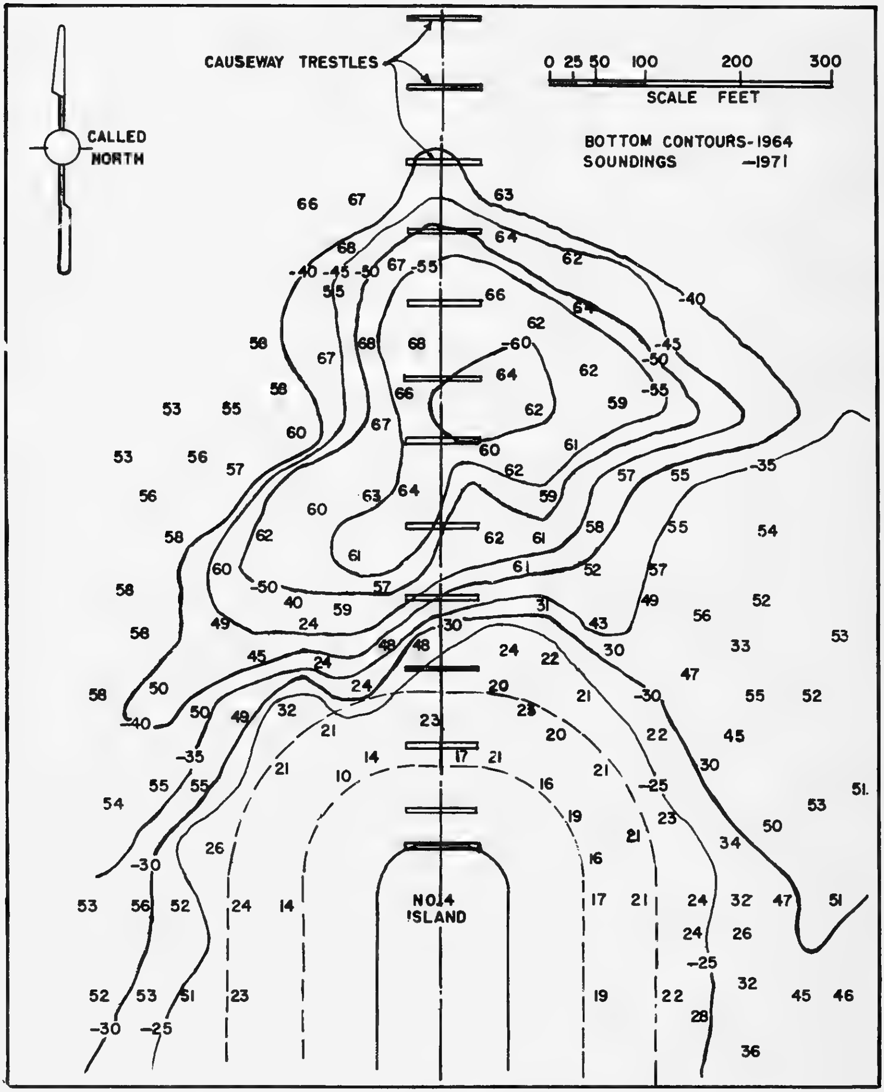

Figure 5. Scour conditions at Island No. 4 (1964-71) (Chesapeake Bay Bridge and Tunnel District).

Table 1. Regional Occurrence of Fishes

Common Name	Scientific Name	Jan	Feb	Mar	Apr	May	Jun	July	Aug	Sept	Oct	Nov	Dec
Chesapeake Bay Mouth													
Coastal and Marine Fishes													
Whiting	*Menticirrhus americanus*						x	x	x	x	x		
Spotted sea trout	*Cynoscion nebulosus*										x	x	
Cobia	*Rachycentron canadum*						x	x	x	x			
Weakfish	*Cynoscion regalis*				x	x	x	x	x	x	x		
Spot	*Leiostomus xanthurus*								x	x	x		
Summer flounder	*Paralichthys dentatus*				x	x	x	x	x	x	x	x	x
Croaker	*Micropogon undulatus*				x	x	x	x	x	x	x		
Tautog	*Tautoga onitis*	x	x	x	x	x	x	x	x	x	x	x	x
Spadefish	*Chaetodipterus faber*						x	x	x	x			
False albacore	*Gymnosarda allcterata*						x	x	x	x	x		
King mackerel	*Scomberomorus cavalla*					x	x	x	x	x	x		
Sheepshead	*Archosargus probatocephalus*					x	x	x	x	x	x		
Small bluefish	*Pomatomus saltatrix*					x	x	x	x	x	x		
Striped bass	*Monrone saxatilis*	x	x	x	x	x	x	x	x	x	x	x	x
Anadramous Fishes													
Menhaden	*Brevoortia tyrannus*	x	x	x							x	x	x
American shad	*Alosa sapidissima*	x	x	x							x	x	x
Hickory shad	*Alosa mediocris*	x	x	x							x	x	x
Glut herring	*Alosa aestivalis*	x	x	x							x	x	x
Alewife	*Alosa pseudoharengus*	x	x	x							x	x	x
Catadramous Fishes													
American eel	*Anguilla rostrata*	x	x	x	x	x	x			x	x	x	x
Dam Neck Disposal Area													
Spanish mackerel	*Scomberomorus maculatus*								x	x	x		
Cobia	*Rachycentron canadum*						x	x	x	x			
Tautog	*Tautoga onitis*	x	x	x	x	x	x	x	x	x	x	x	x
False albacore	*Gymnosarda alleterata*								x	x	x	x	
King mackerel	*Scomberomorus cavalla*									x	x	x	
Sheepshead	*Archosargus probatocephalus*						x	x	x				
Bluefish	*Pomatomus saltatrix*					x	x	x	x	x	x	x	
Striped bass	*Morone saxatilis*	x	x	x	x	x	x	x	x	x	x	x	x
Black drum	*Pogonias cromis*									x	x		
Common skate	*Raja erinacea*	x	x	x	x	x	x	x	x	x	x	x	x
Cownose ray	*Rhinoptera bonasus*	x	x	x	x	x	x	x	x	x	x	x	x
Spiny dogfish	*Squalus acanthisa*				x	x	x						

(U.S. Army Engineer District, Norfolk, 1973).

Table 2. Biota Common to Project Area

Common Name	Scientific Name
Benthic Species	
Plant Benthos	
Sea lettuce	*Ulva lactuca*
Sea lettuce	*Enteromorpha intestinales*
Animal Benthos	
Sand dollar	*Mellita quinquesperforata*
Common starfish	*Asterias forbesi*
Sea cucumber	*Thyone briareus*
Sea urchin	*Arbacia punctata*
Clam worm	*Nereis limbrata*
"Flowered worm"	*Hydroides hexagonus*
Quahog	*Mercenaria mercenaria*
Razor clam	*Esis directus*
Blue crab	*Callinectes sapidus* (female only)
Jonah crab	*Cancer borealis*
Rock crab	*Cancer irroratus*
Spider crab	*Libinia emarginata*
Sea squirt	*Molgula manhattensis*
Summer flounder	*Paralichthys denatus*
Macrozooplankton and Invertebrate Nekton	
Planton	
Comb jelly	*Mnemiopsis leidyi*
Stinging nettle	*Chrysaora quinquecirrha*
Lion's mane jelly	*Cyanea capillata*
Skeleton shrimp	*Caprella acutifrons*
Nekton	
Arrow worm	*Sagitta elegans*
Ribbon worm	*Cerebratulus lacteus*
Prawn	*Palaemonetes vulgaris*
Squid	*Loliguncula brevis*
Waterfowl	
Black duck	*Anas rubripes*
Ruddy duck	*Oxyura jamaicemsis*
Surf scoter	*Melanitta perspicillata*
Black scoter	*Oidemia nigra*
Brant	*Branta bernicla*
Open Water, Shore, and Wading Birds	
Snowy egret	*Leucophoyx thula*
Tricolored heron	*Hydranassa tricolor*
Double-crested cormorant	*Phalacrocorax auritus*
Gannet	*Moris bassana*
Wilson's petrel	*Oceanites oceanicus*
Herring gull	*Larus argentatus*
Laughing gull	*Larus atricilla*
Forster's tern	*Sterna forsteri*
Common tern	*Sterna hirundo*

(U.S. Army Engineer District, Norfolk, 1973).

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2. Permanent Drilling Islands.

a. Long Beach, California—THUMS Islands.

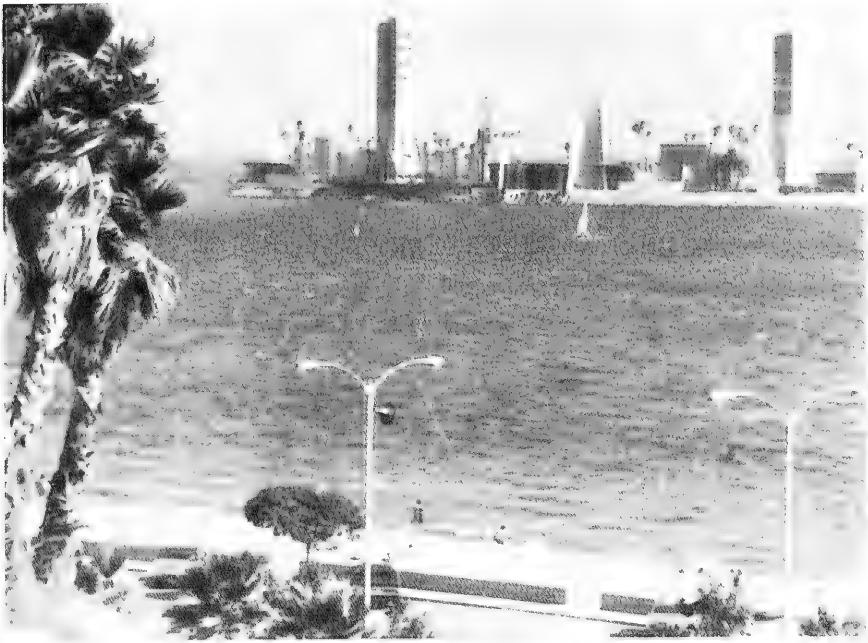

Figure 6. View of Island White from Long Beach waterfront (August 1973).

(1) Construction.

(a) Completed. March 1967.

(b) Type. Armor rock perimeter with dredged sand-filled core (Fig 6).

(c) Purpose. Support of offshore oil-drilling operations.

(2) Owner. Department of Oil Properties, City of Long Beach, Long Beach, California 90802 (as Trustee for State of California).

(3) Field Contractor. THUMS Long Beach Company, 840 Van Camp Street, Long Beach, California 90802 (combination of Texaco, Humble, Union, Mobil, and Shell Oil Companies).

(4) Location (approximate). Long Beach Harbor, San Pedro Bay, Pacific Ocean; at various distances from 400 to 2,300 yards offshore.

Latitude: 33°45′N. – Longitude: 118°10′W.

(5) Physical Environment.

(a) Protected Ocean Environment. Proximity to shore eliminates danger of ocean-generated waves from north and west. Long Beach Harbor breakwaters provide

protection on the south and southwest; open exposure to storms and waves approaching along coast from southeast. Island Chaffee (almost completely exposed) affords some protection from the southeast for the other islands (Fig. 7).

(b) Wave Conditions. Maximum 6-foot breaking waves from south. Those few times when 20-foot waves have reportedly occurred at the harbor breakwater, up to 7-foot waves have been reported at Island Chaffee.

(c) Currents. Variable tidal currents.

(d) Winds. Prevailing west-southwest winds from April through September; and west winds from October through March, with maximum recorded velocity of 54 knots during this period.

(e) Storm Surge and Astronomical Tides.

MHHW, 5.3 feet above MLLW

Mean tide level, 2.7 feet above MLLW

Extreme low, 2.5 feet below MLLW

(f) Littoral Transport. Negligible.

(g) Water Depth at Structures. From 25 to 40 feet at MLLW.

(h) Foundation Conditions. Sandy bay bottom.

(6) Structural Features (Figs. 8 and 9).

(a) Dimensions of Basic Structures.

1 Area at Working Level (approximate). Grissom, 8.8 acres; White, 10.0 acres; Chaffee, 10.0 acres; and Freeman, 10.0 acres.

2 Side Slope. Armor rock, 1:1.5, and others (Fig. 8).

3 Finished Grade at Working Level. 15 feet above MLLW.

(b) Unusual Features. Prime example of possibility of combining functional adequacy with pleasing aesthetic appearance during construction and operations phases.

(7) Instrumentation. Special flood monitoring tests and pressure surveys are routinely made. The elevation of more than 350 new bench marks is established twice a year. Until February 1972, this was a quarterly procedure. Recording tide-gage stations are located on two of the islands to serve as a check on leveling from shore. Measurements of horizontal movements are also monitored by use of a geodimeter. Reports on surface stability are prepared and distributed twice yearly.

Special subsidence and compaction monitoring tools were developed and are used, but mainly in connection with the drilling operations.

(8) Structural Performance.

(a) Performance. Excellent.

(b) Maintenance. Maintenance work on the island structures has been kept to an absolute minimum by regularity of inspection and immediate repair of all armor rock areas (over 60,000 tons of material in 1972). Settlement in work areas has been localized and very minor, occurring mostly on Island White. Since this was the first island to be completed, it is

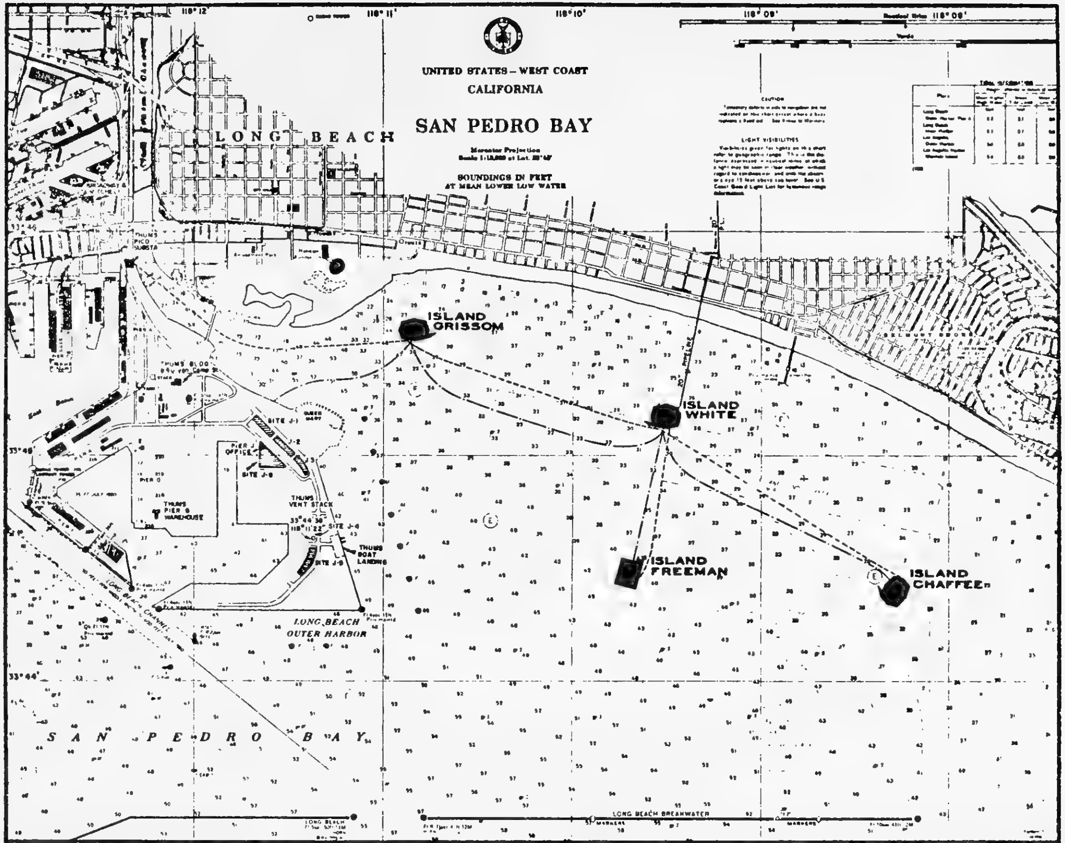

Figure 7. Location plan (THUMS Long Beach Co.).

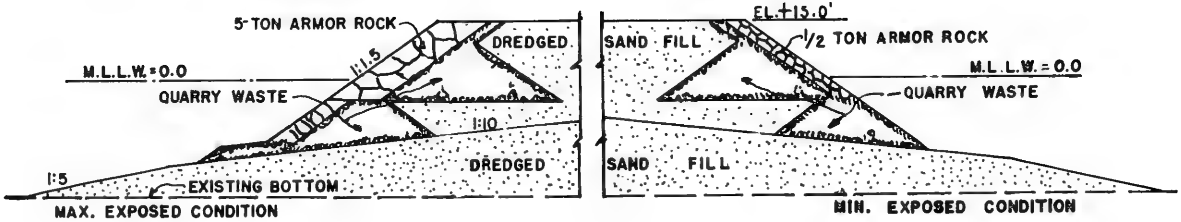

Figure 8. Typical sections—armor protection.

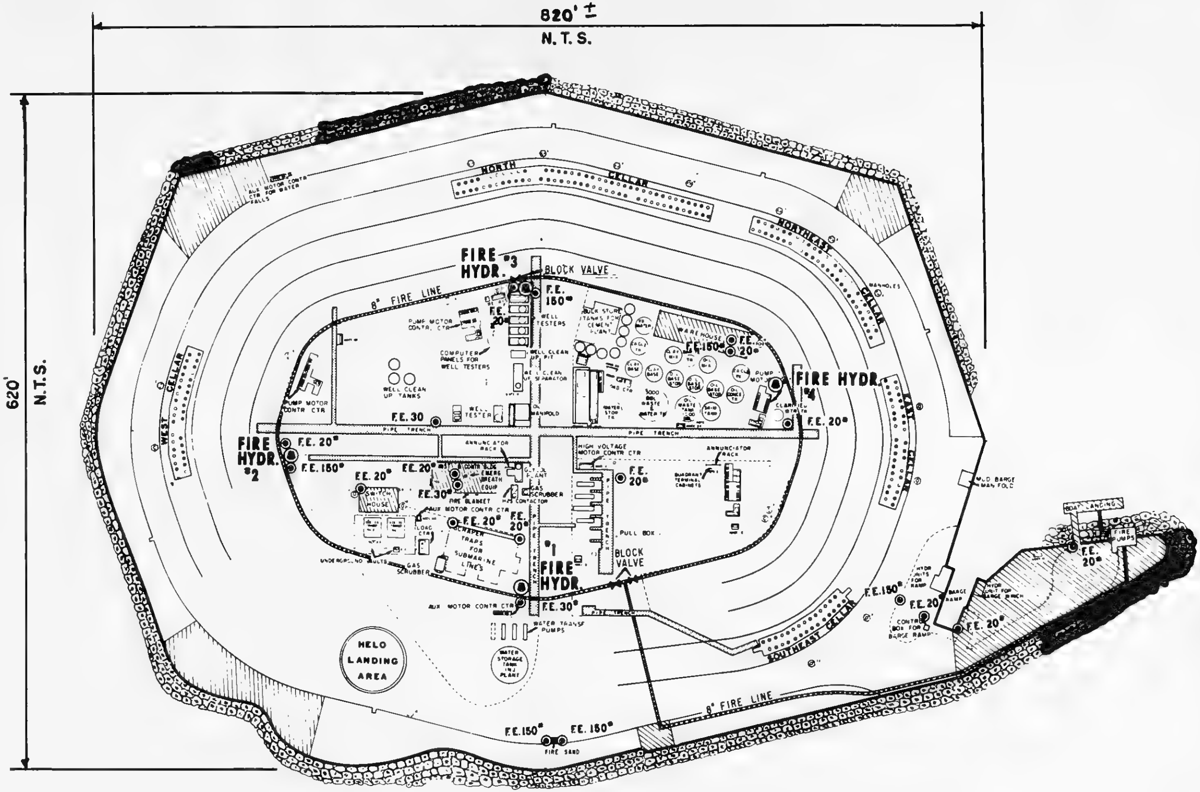

Figure 9. Typical plan view of Island Grissom (THUMS Long Beach Co.).

probably due to poor compaction methods used during construction. Since construction, wave conditions in Long Beach Harbor have dislocated sections of armor rock. In repairing these sections, rock sizes have been increased, the largest being 8 tons instead of the original 5 tons. These replacement sections are proving to be more satisfactory than the original design.

(9) Effect of Structure on Environment.

(a) Physical. Although ocean-generated waves can enter the harbor unobstructed from the southeast, they cause reflected waves in various directions within the harbor up to 6 feet.

(b) Biota. Heavy marine growth on armor stone, attracting fish and sport fishermen to immediate areas of the islands.

(c) Aesthetics. Since the THUMS drilling islands are directly offshore from the civic center and beaches, aesthetics are of importance. City requirements for the THUMS project were probably the most stringent ever imposed on the oil industry, even inducing voluntary moves by the industry for maintaining the natural appearance of the area. Screens, lighting, free-form structures, landscape planting, and artificial waterfalls were built to serve the double purpose of beautification and concealment of drilling and producing operations (Fig. 10). Island-producing facilities are above ground, but all wellheads are recessed in multiple-well cellars.

As a result of these efforts, the islands have won two awards for engineering and aesthetic design. Local feeling is that the drilling islands enhance the harbor view rather than detract from it.

(10) Engineering.

(a) Engineering Design. Moffatt and Nichol, Engineers, 250 W. Wardlow Road, Long Beach, California 90807.

(b) Aesthetic Design. Linesch and Associates, 320 Bixby Road, Long Beach, California 90807 (formerly Linesch and Reynolds).

(11) Construction Contractors. Connally Pacific Company, 1925 Water Street, Long Beach, California 90802.

(12) Construction Date. June 1965 to March 1967 (four islands).

(13) Construction Cost. Cost breakdown for construction of a typical 10-acre island from THUMS Long Beach Company:

(a) Dredge fill, 1,000,000 cubic yards at \$0.70 per cubic yard,	\$ 700,000
(b) Quarry rock, 80,000 cubic yards at \$6.25 per cubic yard,	500,000
(c) Armor rock, 35,000 tons at \$5.14 per ton,	180,000
(d) Sheet piling, dolphins, etc.,	390,000
(e) Gravel cover, engineering, etc.,	130,000
Total	\$1,900,000

Figure 10. Typical view of island shoreline showing landscaping along top of armor protection and buildup of encircling berm to conceal most of the operational activity from the mainland. Note the camouflage of main drill derricks in decorative towers (August 1973).

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b. Punta Gorda, California–Rincon Island.

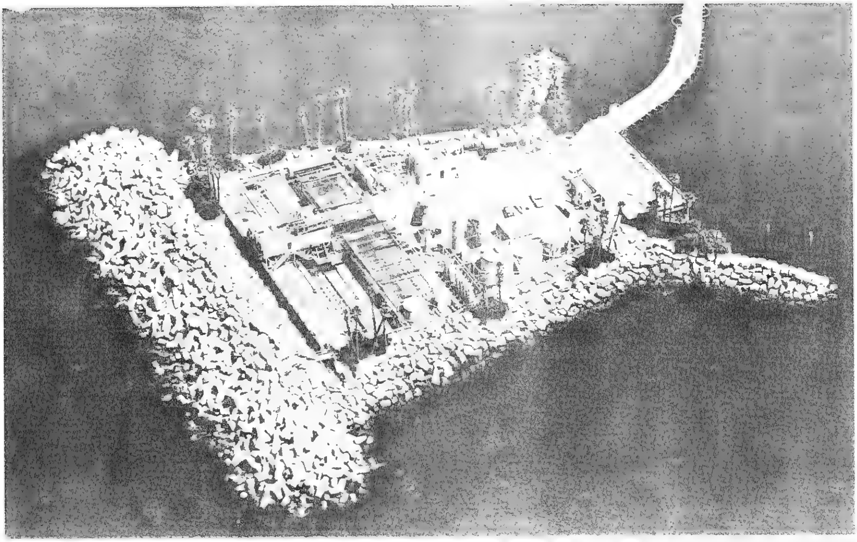

Figure 11. Aerial oblique view of Rincon Island.

(1) Construction.

(a) Completed. September 1958.

(b) Type. Fill-type artificial island with sand core enclosed by armor rock and tetrapods weighing up to 31 tons each (Fig. 11).

(c) Purpose. Support of offshore oil-drilling operations.

(2) Owner. Atlantic Richfield Company, 515 S. Flower Street, Los Angeles, California 90017 (formerly Richfield Oil Corporation).

(3) Location (approximate). Pacific Ocean in Santa Barbara Channel, about 2,700 feet south of Punta Gorda, California.

Latitude: 34°21′N. – Longitude: 119°27′W.

(4) Physical Environment (Fig. 12).

(a) Protected Ocean Environment. Location in Santa Barbara Channel provides some protection from north by California mainland and from south by Channel Islands with reduction of energy of ocean-generated waves.

(b) Wave Conditions. Both sea and swell conditions fairly constant throughout year, originating in northwest quadrant with about 50 percent from northwest. Channel Islands have little effect on this exposure.

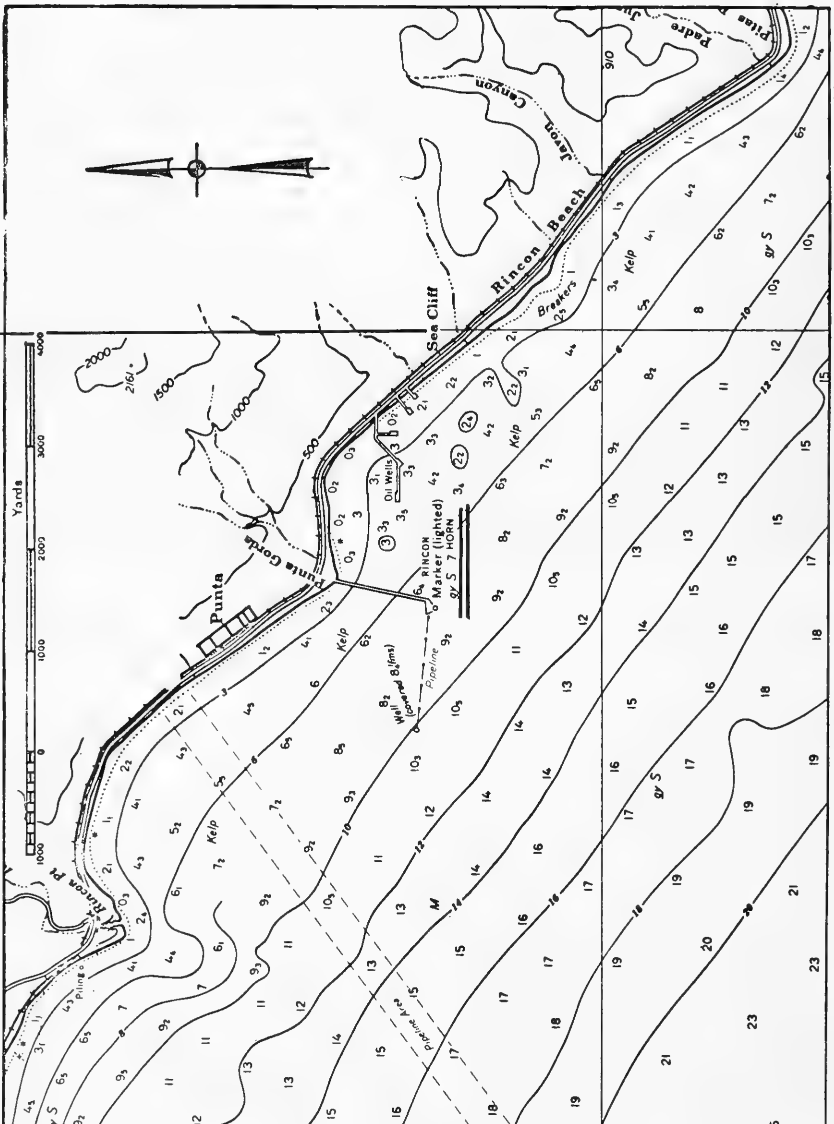

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Figure 12. Location plan.

(c) Currents. Variable in direction (depending to great extent on wind), with weak nontidal flow, setting eastward in spring and summer and westward in fall and winter.

(d) Winds. Prevailing winds westerly with southeasterly storms occurring during winter.

(e) Storm Surge and Tides.

Extreme high, 7.50 feet above MLLW
MHHW, 5.40 feet above MLLW
MSL, 2.58 feet above MLLW
Extreme low, 2.50 feet below MLLW

(f) Littoral Transport. Appreciable in the general area but negligible at the site.

(g) Water Depth at Structure. 48 feet (MLLW) at deepest point.

(h) Foundation Conditions. Overburden of silty sand ranging into sandy silt from 14 to 25 feet thick at island; average slope 3 percent with shale or siltstone formation below. Faults are known to exist near the site, most of them inactive.

(5) Structural Features (Figs. 13 and 14).

(a) Dimensions of Basic Structure.

1 Area (approximate). Bottom, 6.3 acres; MLLW, 3.2 acres; working level, 2.1 acres; and working level (usable), 1.1 acres.

2 Side Slopes. 1.5 on 1.

3 Finished Grade at Working Level. 16.0 feet above MLLW.

(b) Unusual Features. The unusual configuration developed from the attempt to obtain best wave protection (Fig. 15). To obtain the most economical but adequate design, extensive studies were conducted to determine most suitable configuration and orientation. The peculiar, partially protected aspect of the site seemed only to introduce additional factors for consideration in these studies and led to the concept of the more protective seaward face to resist the larger waves.

(6) Design Data.

(a) Design Conditions.

Depth at structure, 41.0 to 48.0 feet at MLLW
Extreme high tide, 7.5 feet at MLLW
Maximum depth, 55.5 feet at MLLW
Design wave, west face, 27.0 feet at MLLW
Design wave, north and south faces, 12.0 feet at MLLW

(b) Model Study. Model studies were conducted at the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi in two series. The first involved a three-dimensional model test to check the island configuration, plus the baffling necessary to maintain quiet water in the small boat harbor on the leeward side of the island and the feasibility of using two concrete ship hulls as a separate submerged breakwater to seaward. The second series involved a two-dimensional model of the proposed revetment on the seaward side.

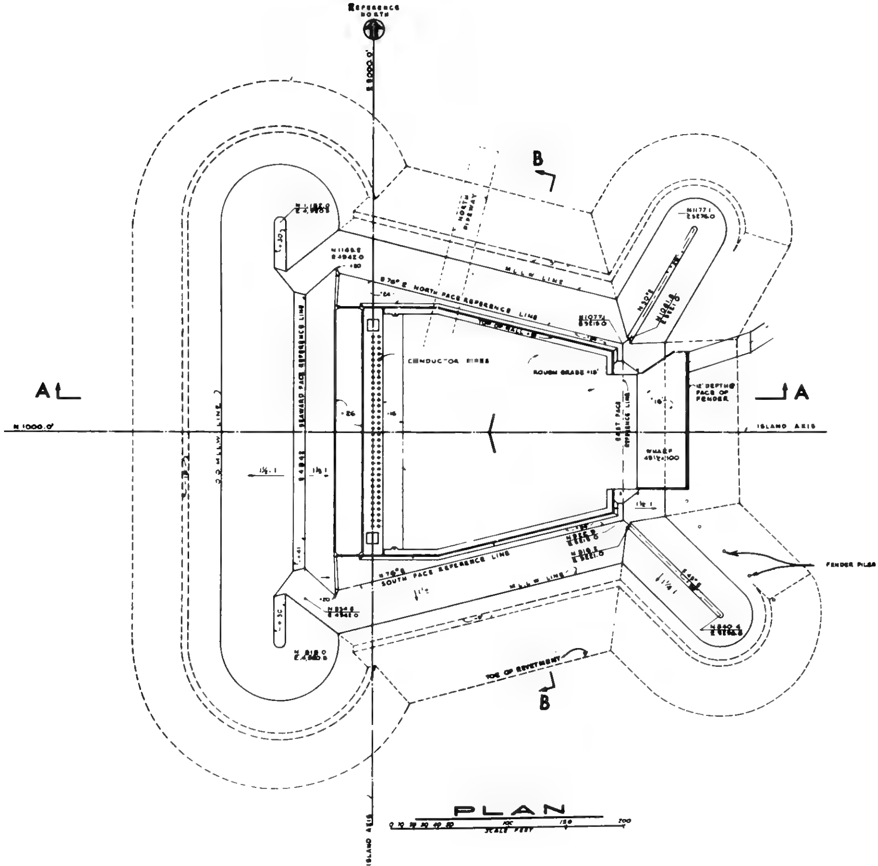

Figure 13. Plan of Rincon Island.

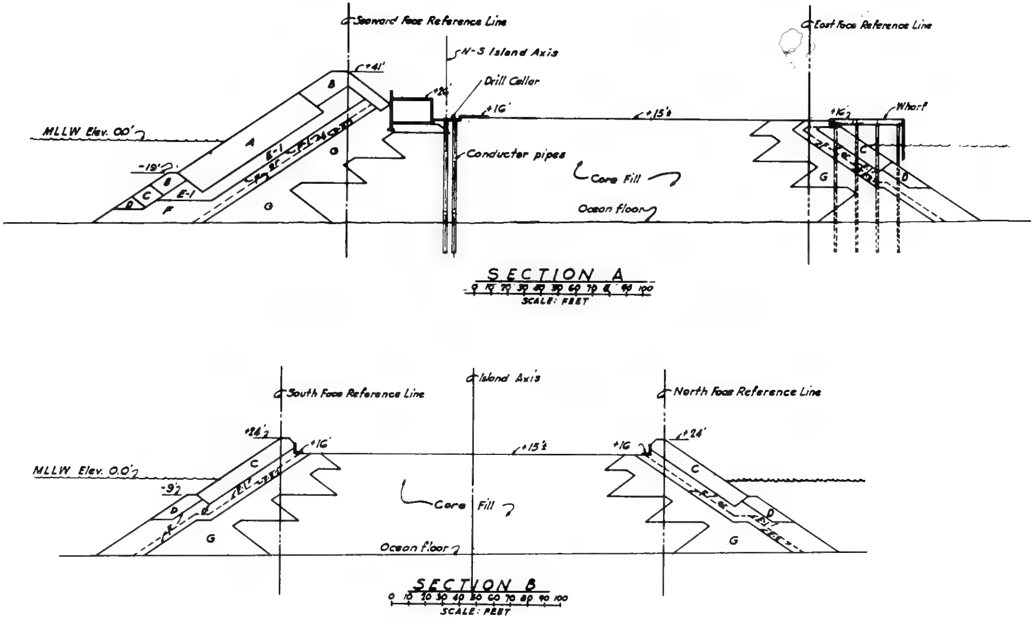

Figure 14. Rincon Island sections.

Figure 15. View of north side of island showing unusual configuration and good condition of armor rock (August 1973).

The study basically confirmed the original design. When increasing the wave height to 34 feet rather than the original design wave of 27 feet, the section showed no tendency to catastrophic failure due to a single wave, consistent with the basic design objective.

(c) Instrumentation. There is no instrumentation set up to record physical conditions at the site.

Various instrumentation installations in the area have been unsuitable for studying Rincon due to distance and type of data recorded.

A group of level check points has been developed on the island from which to run a releveling program periodically and to tie in with bench marks on shore maintained by the Earth Science Laboratories of the National Oceanic and Atmospheric Administration (NOAA).

(7) Structural Performance.

(a) Performance. Structure performing satisfactorily. Lack of recorded data leaves uncertainty as to maximum condition experienced. Eyewitness evidence indicates 20 feet may have been the highest wave experienced. As designed, the armor would have proved unstable for waves over 27 feet and overtopping would have occurred with 34-foot waves. Neither of these conditions has occurred.

Settlement during the construction period was approximately the expected 6 inches. Since then there has been no change in the island elevation relative to the onshore bench marks.

A crack has developed recently in the end wall of a concrete block building on the filled area at the east end of the island (causes undetermined to date).

(b) Maintenance. Regularly scheduled inspections are made of the condition of the armor both on the surface and underwater; levels are run at regular intervals, checking elevations at selected points against onshore bench marks. Good results have been obtained.

Maintenance requirements for the structure have been nominal.

(8) Effect of Structure on Environment.

(a) Physical. Because of considerable littoral transport along the mainland coastline in the area, it was expected that the island's size and location would keep interference with littoral transport to a minimum. Aerial photos show no noticeable changes in the adjacent shorelines.

A slight variation in bottom contours has developed, partly due to construction operations, and partly to the island's interference with the natural easterly channel flow.

(b) Biota. Assistance (in the form of physical facilities and complete cooperation) has been provided by the owner to Moorpark College, Moorpark, California for a study of marine life in the area.

A recent study (Keith and Skjei, 1974) was undertaken to determine the changes brought on by the island construction in the environment for the marine biota in the area. The results indicate "the development of a mature and balanced reef out of an area which

might well have been considered to be a biological desert." Where the number of species before construction probably was 25 to 30, since construction, 298 have been recorded, representing all major marine forms of plant and animal life.

(c) Aesthetics. Owner went to considerable expense to achieve a natural appearance for the structure and met with good success. Oil facilities are a common sight in the area, but Rincon Island has had none of the unfavorable comment as some of the other nearby installations.

(9) Engineering.

(a) Design. John A. Blume and Associates, Engineers, 130 Jessie Street, San Francisco, California 94105.

(b) Wave Prediction. Paul Horrer, San Diego, California 92101.

(c) Revetment Stability. Robert Y. Hudson, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi 39180.

(10) Construction Contractor (island only). Guy F. Atkinson Company, San Francisco, California 94101.

(11) Construction Date. Contract awarded August 1956; site work February 1957 to August 1958.

(12) Construction Cost. Approximately $3,250,000 (for island only).

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c. Seal Beach, California.

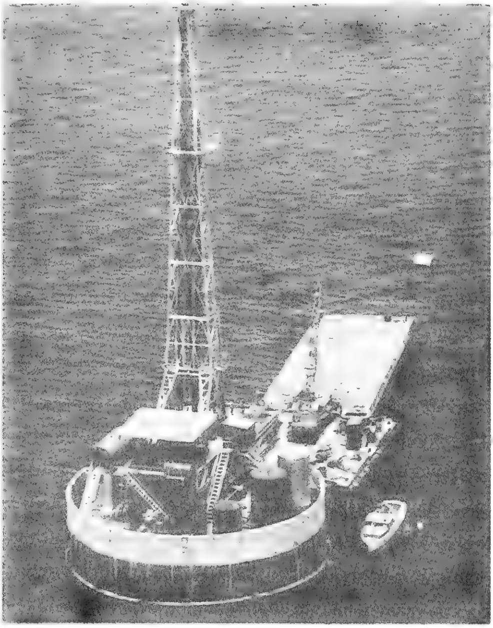

Figure 16. Seal Beach—permanent drilling island (Courtesy of Oil and Gas Journal).

(1) Construction.

(a) Completed. 1954.

(b) Type. Sand and rock-filled steel sheet-pile cell with riprap outer protection (Fig. 16).

(c) Purpose. Support of offshore oil-drilling operations.

(2) Owners. Humble Oil and Refining Company (Manager), 1800 Avenue of the Stars, Los Angeles, California 90067, and Texaco, Incorporated, 3350 Wilshire Boulevard, Los Angeles, California 90005. (Monterey-Texas Company, a joint venture of Monterey Oil Company and Texaco, Incorporated, original owners.)

(3) Location. Pacific Ocean, about 1 mile south of channel breakwater for Alamitos Bay at Seal Beach, California.

Latitude: 33°43′20″N. – Longitude: 118°07′30″W.

(4) Physical Environment (Fig. 17).

(a) Protected Ocean Environment. Protected from northern quadrants by mainland and Long Beach Harbor breakwaters. Otherwise exposed to ocean winds and waves.

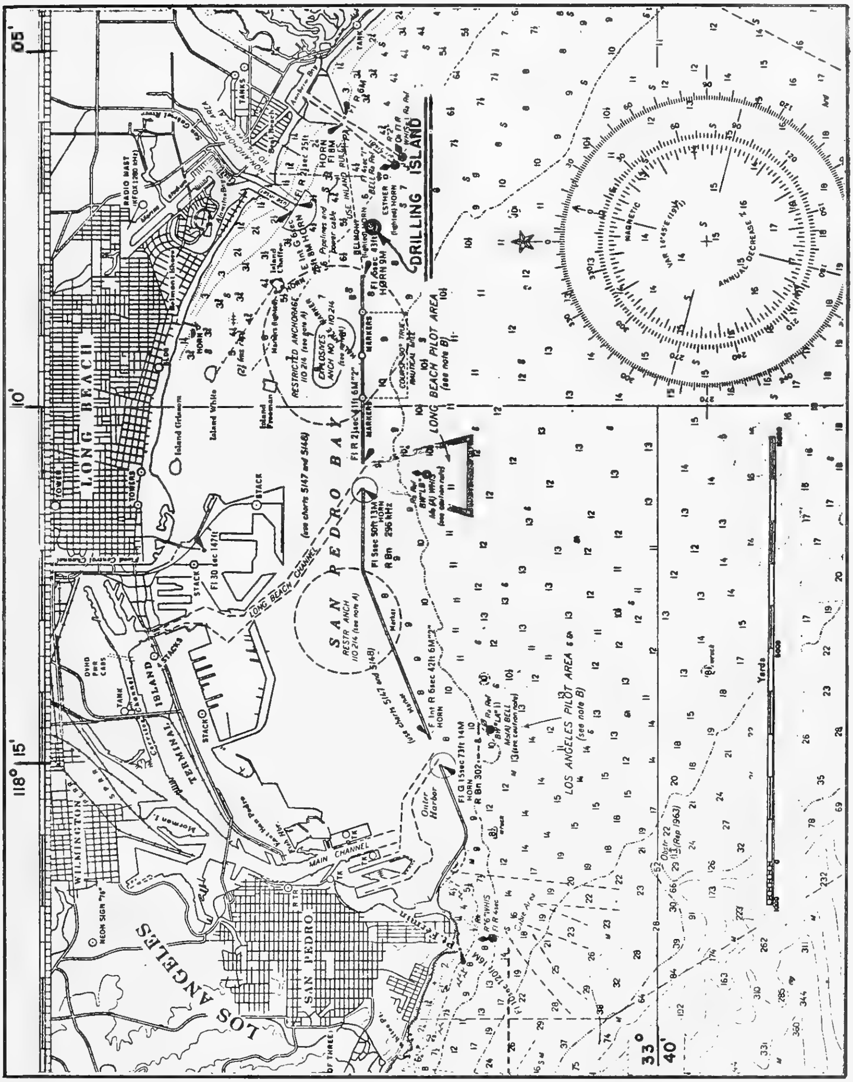

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Figure 17. Location plan.

(b) Wave Conditions. Both sea and swell conditions, fairly constant throughout year, originating in northwest quadrant with about 50 percent from northwest, in the low to medium range.

(c) Currents. Variable in direction but mostly north-northwest and south-southeast and averaging 1/3 to 2/3 knots.

(d) Winds. Prevailing wind direction during most of year is northwest, with low average velocity. Occasional east and northeast winds through coastal mountains with speeds possibly exceeding 50 miles per hour. Gales infrequent, occurring in only 1 percent observations in December and January, with no predominant direction.

(e) Storm Surge and Tides. Maximum astronomical tide range about 10 feet; storm surge not known but reportedly in range of additional 10 feet.

(f) Littoral Transport. Reportedly negligible.

(g) Water Depth at Structure. 42 feet MLW.

(h) Foundation Conditions. Sandy ocean bottom.

(5) Structural Features (Figs. 18 and 19).

(a) Dimensions of Basic Structure. 75-foot-diameter steel sheet-pile cell with 58.5- X 72.5-foot pile-supported wharf adjoining.

(b) Unusual Features. Earliest of California's offshore drilling islands designed to satisfy State Lands Commission resolution requiring wells in area to be drilled from filled islands offshore.

(6) Design Data.

(a) Design Conditions:

Depth of structure, 42 feet MLLW
Astronomical tide, 5.4 feet
Storm surge (not available)

(b) Model Study. None.

(c) Instrumentation. None.

(7) Structural Performance.

(a) Performance. Satisfactory with no measurable movement. In 1972, during a general overhaul program, test sections cut from sheeting indicated negligible decrease in thickness due to corrosion, and core borings through concrete top surface of cell indicated less than 1 inch average settlement of filled interior.

(b) Maintenance. No maintenance other than repair of timber fendering. In 1972, a general overhaul consisted primarily of sandblasting and painting of all exposed steel sheet-pile surfaces above water. All wharf timber piles, gunited before driving, were checked for soundness and repaired with concrete as necessary, mostly at top just below pile cap.

(8) Effect of Structure on Environment.

(a) Physical. No known physical effect on hydrography in the area.

(b) Biota. Marine growth on the underwater, unpainted areas attracts fish which in turn attract sport fishermen (Fig. 19).

(c) Aesthetics. First platform in area; built before aesthetic appearance was a major consideration.

(9) Engineering (Pyles, 1955). Capt. George F. Nicholson, Long Beach, California.

(10) Construction Contractor. Healy Tibbitts Construction Company, 1400 W. 7th Street, Long Beach, California 90813.

(11) Construction Date. 1952 to 1954 (with delays due to litigation).

(12) Construction Cost. Approximately $1,000,000 for filled cell.

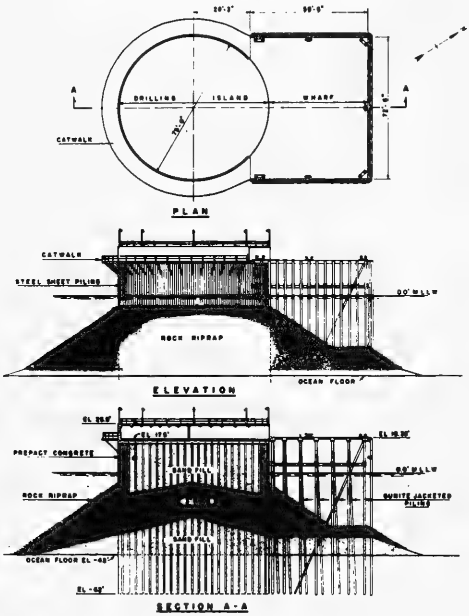

Figure 18. Plan and cross-sectional drawing of filled island (Pyles, 1955).

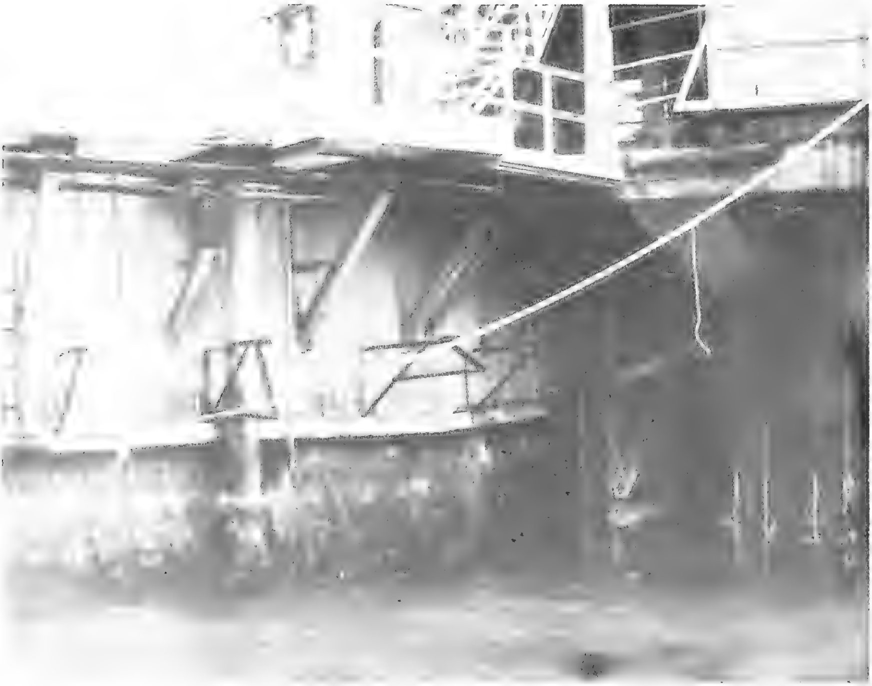

Figure 19. View of steel sheet piling. Note painted area above high tide line, free of marine growth.

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3. Offshore Breakwaters.

a. Sandy Bay, Cape Ann, Massachusetts.

Figure 20. Aerial view of Sandy Bay Breakwater (1967).

(1) Construction.

(a) Completed. June 1916.

(b) Type. Rubble-stone mound substructure surmounted by a sloped stone block superstructure with 20- to 30-ton granite cap stones and 10-ton apron stones (Fig. 20).

(c) Purpose. A harbor of refuge for coastal commercial sailing vessels.

(2) Owner. Department of the Army, U.S. Army Engineer Division, New England, Waltham, Massachusetts 02154.

(3) Location. Atlantic Ocean in Sandy Bay, about 1 mile off north shore of Cape Ann, midway between Boston, Massachusetts and Portsmouth, New Hampshire.

Latitude: 42°40′40″N. – Longitude: 70°35′30″W.

(4) Physical Environment.

(a) Protected Ocean Environment. Harbor and breakwater are protected from westerly through southerly quadrant by mainland. Area is exposed to ocean winds and

waves from northerly through easterly to southerly directions although partially sheltered by shoals to the east (Fig. 21).

(b) Wave Conditions. Fully exposed to deepwater-generated waves from north through east through southeast, except for partial shelter afforded by *Flat Ground* and *Salvages* shoals. Heaviest seas are from the northeast through east with maximum wave heights occurring during December, January and February.

(c) Currents. Tidal currents at site are reported as negligible.

(d) Winds. Strongest and most dangerous winds from northeasterly through easterly direction. Fully exposed to easterly, northeasterly and northerly gales.

(e) Storm Surge and Tides. Approximate extreme high and low tides recorded for the area are 12 feet above MLW and 3.5 feet below MLW. Mean normal tidal range, 8.6 feet; mean spring tidal range, 10.0 feet.

(f) Littoral Transport. No data available.

(g) Water Depth at Structure. Varies from 25 to 75 feet at MLW.

(h) Foundation Conditions. No data available. U.S. Coast and Geodetic Survey Chart No. 243 (Fig. 21) indicates hard, rocky bottom conditions at the breakwater.

(5) Structural Features.

(a) Dimensions of Basic Structure (U.S. Army Engineer Division, New England, 1972).

1 Length. Original project proposed a continuous breakwater 9,000 feet long, extending from Avery Ledge, 3,600 feet (southern arm) to an angle at Abner's Ledge, then 5,400 feet (western arm) toward Andrews Point. As built, the substructure of the southern arm and the first 2,500 feet of the western arm were completed to approximately MLW. A total length of 922 feet of superstructure was completed of which 720.6 feet was on the southern arm and 201.4 feet on the western arm (Fig. 21).

2 Side Slopes (Fig. 22).

a Seaward Slope. 1 on 1 to elevation 8.6 feet; than 1 on 2 to elevation −25 feet; and 1 on 1 below elevation, −25 feet below MLW.

b Shoreward slope. 1 on 1.

3 Crest Elevation and Width (U.S. Army Engineer Division, New England, 1970) (Fig. 22).

a Completed. 22 feet above MLW (13.4 ft. above MHW, and 20 ft. wide).

b Incomplete. Core built up generally to MLW.

(b) Unusual Features. Workmanship and craftsmanship exhibited in breakwater superstructure of uniform-cut granite blocks laid flat and in horizontal courses (Fig. 23).

(6) Design Data.

(a) Design Conditions. Data not available.

(b) Model Study. None.

(c) Instrumentation. None.

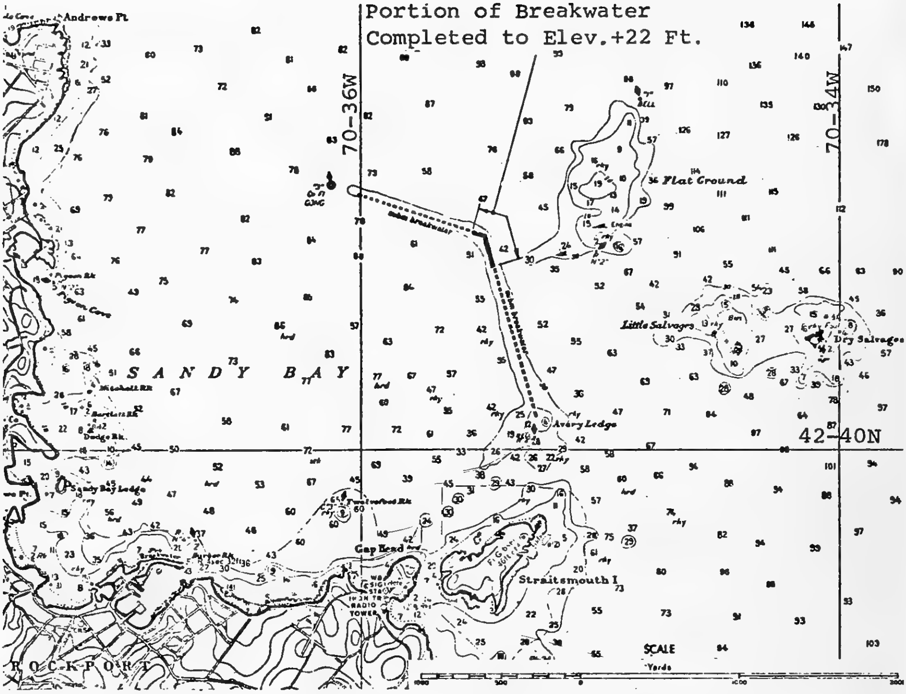

Figure 21. Location plan (U.S. Coast and Geodetic Survey Chart 243).

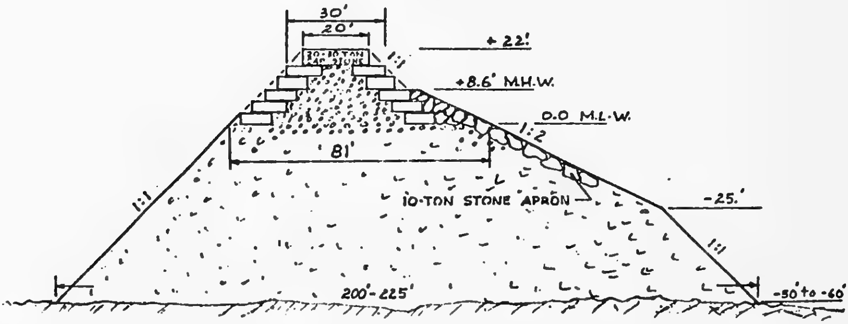

Figure 22. Typical proposed cross section (U.S. Army Engineer Division, New England, 1972).

Figure 23. Looking north along the granite block superstructure of the breakwater (1916).

(7) Structural Performance (U.S. Army Engineer Division, New England, 1972).

(a) Effectiveness of Structure. During construction, it was decided to complete 900 feet of the breakwater and await the test of storms, ice and expansion due to summer heat. With the disappearance of commercial sailing vessels, continuation of the project was not economically justified. The breakwater as completed reduced, to some extent, sea conditions in the bay during easterly storms but had little effect on storms out of the northeast to north. At present, the structure offers little use to commercial or recreational mariners.

(b) Integrity of Structure. Since 1916 the superstructure of the western arm has been almost completely destroyed (75 feet of deterioration between 1968 to 1972). Except for minimal damage at the south end and slight shifting of capstones, the southern arm is in excellent condition. Damage to the western arm is primarily by severe wave attack from the northeast and stopping construction without properly providing temporary end protection. Because of orientation of the structure, northeast waves have less effect on the southern arm.

(8) Effect of Structure on Environment.

(a) Physical. Local townspeople have stated that the partially constructed breakwater is a hazard to navigation and should be either completed or removed. In the

opinion of the New England District, Corps of Engineers, the breakwater is properly marked by navigation aids and does provide some limited protection (U.S. Army Engineer Division, New England, 1966).

(b) Biota. No recorded data obtained. Oral reports indicate large fish populations in immediate area of breakwater.

(c) Aesthetics. Due to distance from shore, breakwater does not affect the aesthetic value of the offshore view; to passing boats, careful workmanship of the superstructure is pleasing to the eye.

(9) Engineering. U.S. Army, Corps of Engineers, Boston, Massachusetts.

(10) Construction Contractor. Information not obtained.

(11) Construction Date. Construction period, 1886–1916.

(12) Construction Cost. 2,133,734 tons of stone placed at a total cost of $1,941,479.

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b. Cape Henlopen, Delaware.

Figure 24. Aerial view southeastward toward Cape Henlopen. Ice piers in foreground and outer breakwater at left; old breakwater close inshore at right (U.S. Army Engineer District, Philadelphia–in press, 1974).

(1) Construction.

(a) Period. 1828–1901.

(b) Type. Rock mound (Fig. 24).

(c) Purpose. Protection of anchorage areas (Breakwater Harbor inside inner breakwater; National Harbor of Refuge inside outer breakwater).

(2) Owner. Department of the Army, U.S. Army Engineer District, Philadelphia, Philadelphia, Pennsylvania 19106.

(3) Location (approximate). Mouth of Delaware Bay, Cape Henlopen, Delaware.
Latitude: 38°49′N. – Longitude: 75°06′W.

(4) Physical Environment.

(a) Protected Ocean-Estuarine Environment. Protected through southwest quadrant by proximity to shore. Except for some protection from waves offered by the tip of Cape Henlopen, fully exposed to the Atlantic to east and northeast, and to full reach of the Delaware Bay to north and northwest (Fig. 25).

(b) Wave Conditions. Heavy seas and swells mainly from the east, reportedly with overtopping occurring during storms.

(c) Currents. Generally east-west tidal current, ranging to about 1 knot in each direction.

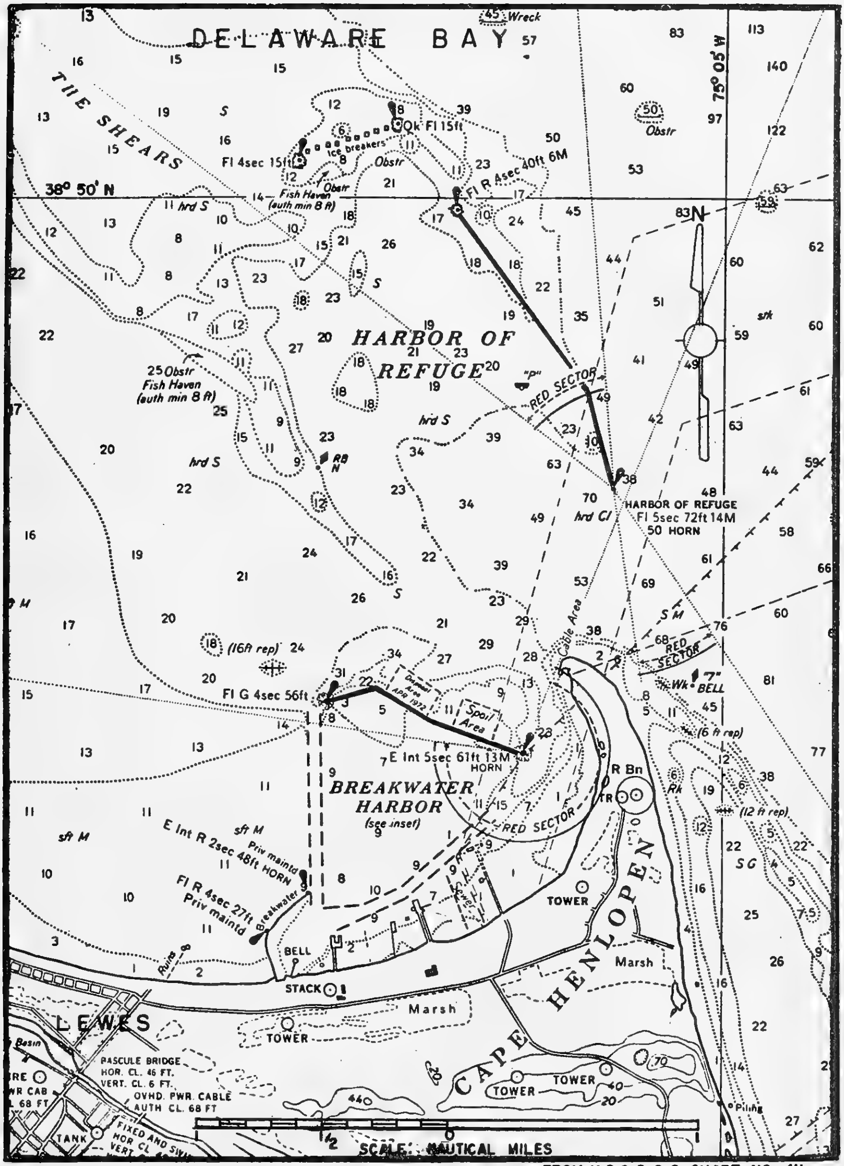

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Figure 25. Location plan.

(d) Winds. Prevailing northwest winds from November through March at speeds up to 12 knots; varying in direction through remainder of year, but not as strong.

(e) Storm Surge and Tides.

Mean tidal range, 4.1 feet above MLW

Spring tidal range, 4.9 feet above MLW

Mean tide level, 2.1 feet above MLW

Extreme high tide, 5.4 feet above MLW

Extreme low tide, –1.1 below MLW

(f) Littoral Transport. Littoral transport has reached a stable condition with little scour or accretion taking place.

(g) Water Depth at Structure. Harbor of Refuge, 10 to 70 feet; Breakwater Harbor, 3 to 23 feet.

(h) Ice Conditions. Thin ice forms on Delaware River in early December; heavier ice from January to March. Tidal currents and heavy traffic of large ocean vessels generally keeps ice broken.

(i) Bottom Conditions. Stable sandy bottom in Harbor of Refuge; shifting silty bottom in Breakwater Harbor (Fig. 25).

(5) Structural Features (Figs. 26 and 27). Stone for the old breakwater was dumped on the bay bottom in a mound averaging 160 feet wide, with side slopes about 45°. In 1833, at the end of five construction seasons, contemporary reports show 75 percent of the breakwater length laid with levels varying from 15 feet above sea bottom to 5 feet above higher water, plus an even greater part of the ice breaker. Work continued intermittently until 1839, when 2,586 lineal feet had been constructed against the 3,600 feet as designed. In 1869 the construction was finally brought to design height of 14 feet above MLW with a width at the crest of 22 feet.

The *Gap* was finally closed from 1882 to 1898 by dumping stone for lower sections, and placing upper stone by derrick barge. An earlier plan had called for a timber bridge and railroad track which was finally discarded for the rubble-mound section.

The outer breakwater, completed in 1901, was constructed basically to the same cross section as the *Gap*, with a length of 8,040 feet at low water, and 7,950 feet at the crest. The seaward side, brought up first, afforded protection for work on the harbor side, and permitted settlement of the lower parts before topping off the seaward face.

(6) Design Data. There are few reliable records to indicate the criteria followed in the original design. The inner breakwater was the first of its type in the Western Hemisphere, but there were precedents in Europe at Cherbourg, Plymouth, and Kingstown. Some records indicate that the inner breakwater was an exact duplicate of the one at Cherbourg, and therefore, greatly over-designed. Recommendations were made at one point to continue construction with the use of stone, which could be safely removed from the old breakwater, although reducing its cross section by as much as 50 percent.

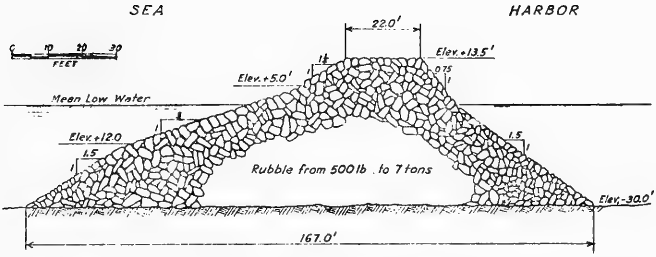


Figure 26. First rock-mound breakwater constructed in the United States—the Delaware breakwater (Quinn, 1972).

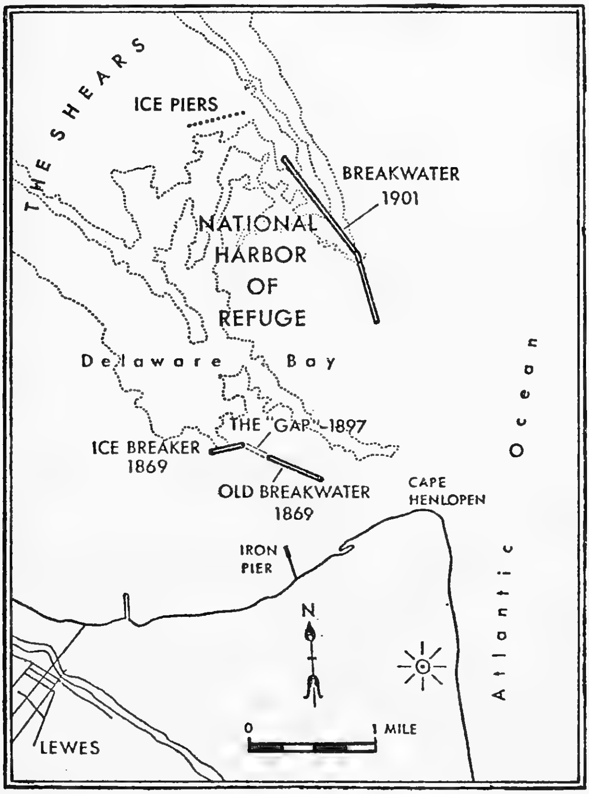


Figure 27. Construction stages (U.S. Army Engineer District, Philadelphia—in press, 1974).

Construction began on the old breakwater in 1828, following the design of William Strickland of Philadelphia and under the general administration of the Quartermaster General, and the technical direction of the Corps of Engineers. Work continued intermittently, depending on need, engineering know-how, and congressional appropriations. The inner breakwater, consisting of the old breakwater and ice breaker (completed in 1869), and the *Gap* (completed in 1897), formed Breakwater Harbor; the outer breakwater completed in 1901, formed the Harbor of Refuge.

During construction, stone sizes were varied to reach a satisfactory size. Specifications in 1828, first called for stones from 200 pounds to 2 tons, but was changed to a 500-pound minimum requirement. Soundings in 1830 indicated a general lowering of the structure "due to wave and tidal action." Further study was undertaken and the size specification revised to 2.25 to 6 tons. Later, stones up to 13 tons were used for the upper parts of the outer breakwater.

By the time design was being undertaken for the outer breakwater, and when the Breakwater Harbor was no longer adequate for the larger, deeper-draft ships, two important changes had taken place: (1) design was now more closely following engineering principles rather than intuition; and (2) construction equipment had now developed to where larger stones could be handled, and precise placing was possible. This permitted certain improvements over the design of earlier sections, although the seaward slope of the existing structure, considered as having gradually settled to a stable position, was maintained in the new design. The height was limited to dissipate the wave forces, without the full impact being absorbed by the structure, and the step arrangement of the top section tended to reduce the scour effect at the toe.

The criteria finally developed for the Delaware Bay work were used as a basis for the Sandy Bay breakwater in Massachusetts and the San Pedro breakwater in California. The simplicity of construction and repair of these breakwaters seemed to outweigh the problem of the massive foundation required for such a structure.

Different aspects and the relation of various stages of construction from some of the engineering drawings and photographs are shown in Figures 28 through 33.

(7) Structural Performance.

(a) Performance. The outer breakwater (Harbor of Refuge), and the inner breakwater (Breakwater Harbor), have both performed exceptionally well, considering their long span of service. Harbor of Refuge, with depths from 15 to 70 feet, affords good protection from the easterly gales; Breakwater Harbor, with depths up to 10 feet is excellent in all weather including the heavy northwesterly gales (U.S. Department of Commerce, 1966).

Records indicate that Breakwater Harbor, more useful in the days of sailing ships, often provided shelter to more than 200 ships at a time.

(b) Maintenance. Settlement and general lowering by wave and tidal action, occurring as expected in the early stages, were corrected during the long construction period. Stability was reached before completion, and since final rock was placed, no maintenance has been necessary.

(8) Effect of Structure on Environment.

(a) Physical. Early theories by engineers that orientation of the breakwaters would cause tidal currents to wear away the Cape and keep the harbor at desired depths, proved false. The Cape has extended in a north and northwesterly direction and Breakwater Harbor has silted from a continually shifting silty bottom condition. Silting of the harbor and the violent tidal action through the *Gap* were important reasons for completing the *Gap* construction.

(b) Biota. No recorded data obtained; verbal reports indicate heavy fish population in area of breakwaters.

(c) Aesthetics. In a generally isolated area, where, probably partly due to *always* having been there, there has been little objection to the sight of the natural rock structures.

(9) Engineering. William Strickland, Consulting Engineer, Philadelphia, Pennsylvania.

(10) Construction Date, Cost and Contractors (Table 3).

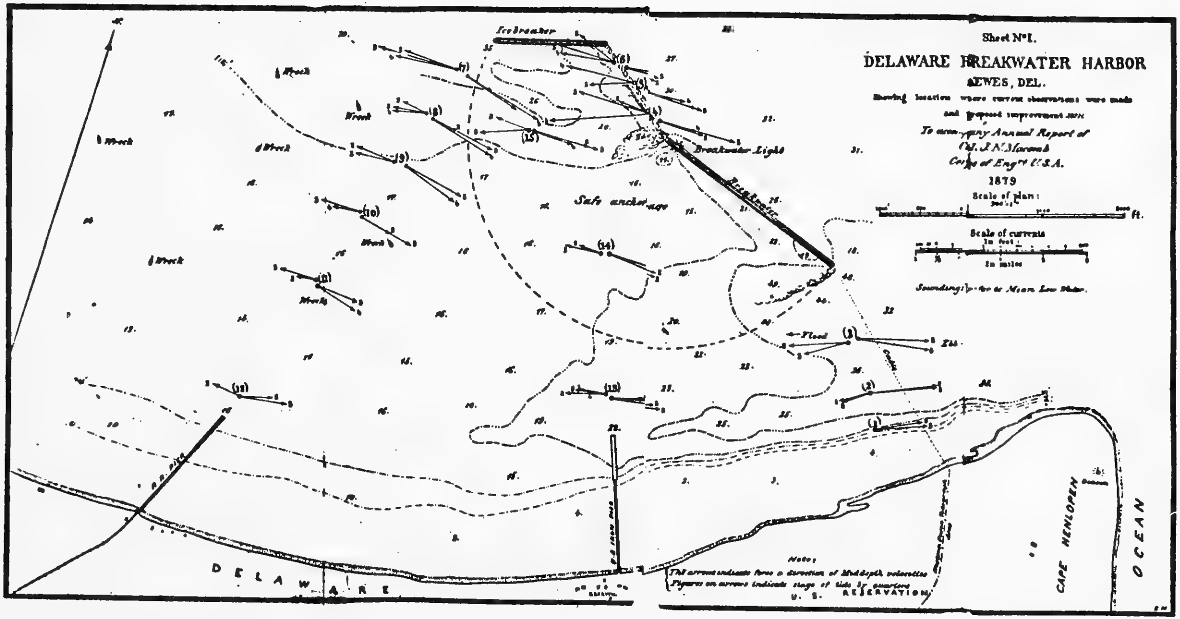


Figure 28. Delaware Breakwater Harbor. Time of current velocity units not identifiable.

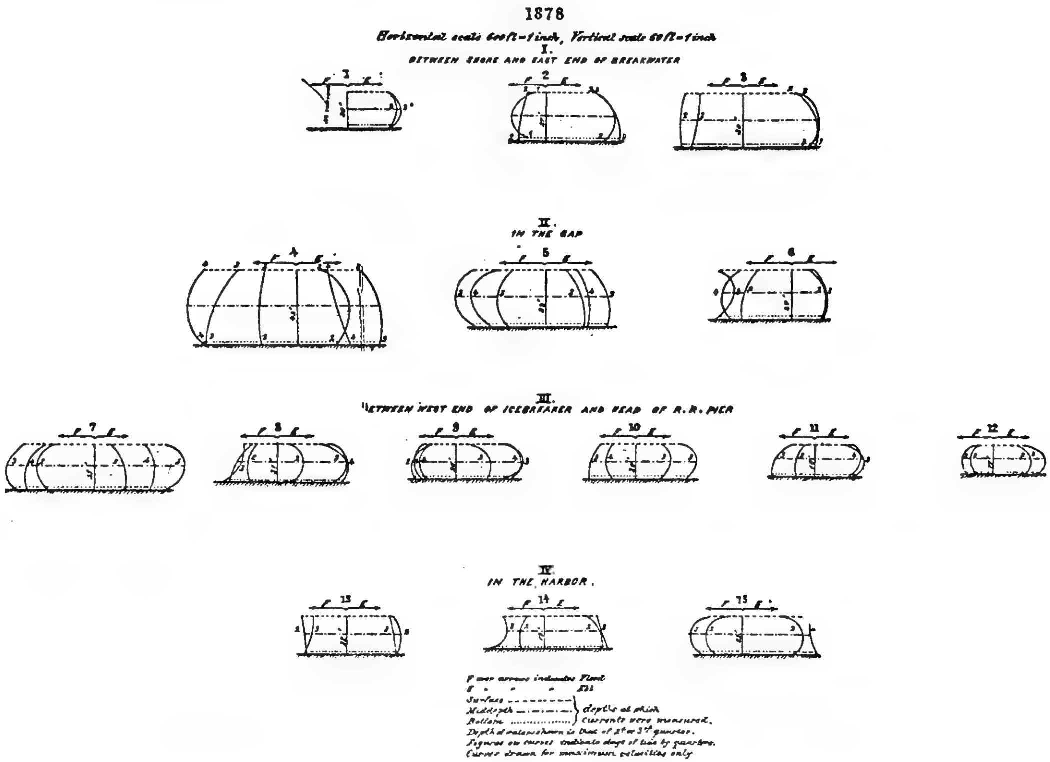
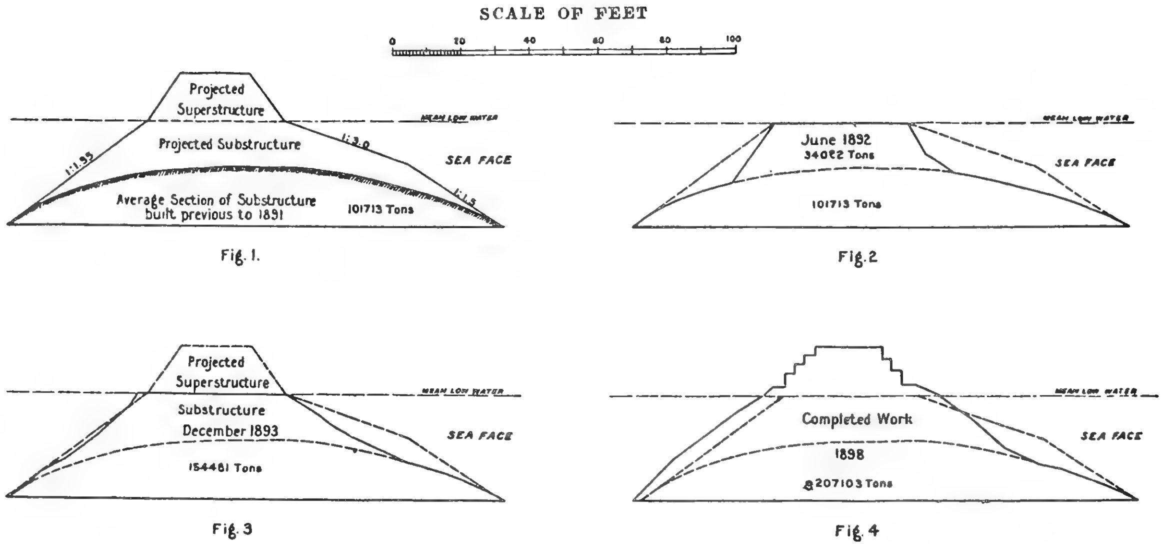


Figure 29. Vertical curves of current velocities, Delaware Breakwater Harbor. Time of current velocity units not identifiable.



Engineer Office, U S Army
Philadelphia, Pa
To accompany final report dated June 19, 1899.

Lieut. Colonel, Corps of Engineers.

Figure 30. Sections of Delaware Breakwater. Drawing prepared for progress report on construction of "Gap."

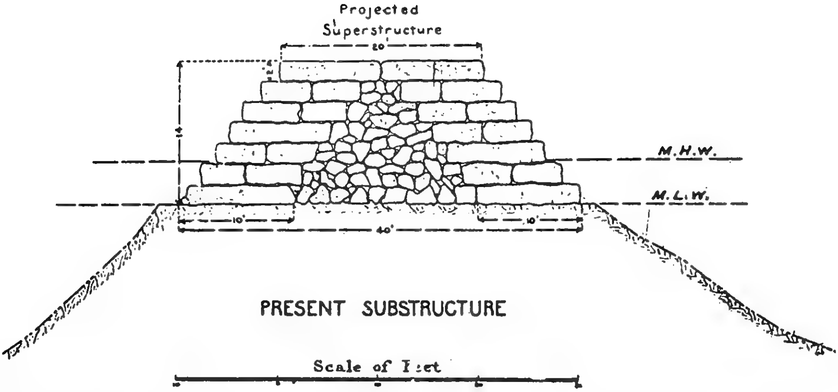


Figure 31. Delaware Breakwater substructure. Drawing was prepared for specifications dated 18 October 1894, showing proposed top structure for the "Gap" (U.S. Army Engineer District, Philadelphia–in press, 1974).

Figure 32. View northward along top of "Gap" construction with ice breaker in background (photo taken in 1890's). Note relatively calm water on harbor side (U.S. Army Engineer District, Philadelphia–in press, 1974).

Figure 33. View southward along harbor side of "Gap" construction (photo taken in 1890's) (U.S. Army Engineer District, Philadelphia—in press, 1974).

Table 3. Summary of Construction Data

Date	Work Done	Contractor	Cost to Date	Tonnage to Date
Old Breakwater and the Ice Breaker				
1828	Construction started	Not known	---	---
1829		Halsey Rogers Company, N.Y.	---	---
1830	242,770 tons deposited	Canvass White & Co., N.Y.	---	---
1831		Leiper & Crosby, Pa.	---	---
1832		15 separate small contractors	---	---
1833	154,459 tons deposited in best progress to date	Leiper, Hill & Jacques, Pa.	---	518,733
1834	122,995 tons deposited	Leiper & Co., J.F. Hill, Pa.	---	640,520
1839	Last load of original construction period, 53 tons deposited, 6 Sept. 1939	Leiper & Co., Pa.	$1,880,000	835,000
1851–1869	Completion to design height but excluding *Gap*		2,123,000	---
Gap Closure				
1882–1898			529,900	
Breakwater and Ice Piers				
1896–1901			2,090,800	
Total Cost Complete Structures			**$4,743,700**	

(U.S. Army Engineer District, Philadelphia—in press, 1974)

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c. Venice, California.

Figure 34. Aerial oblique photo of Venice Breakwater (Inman and Frautschy, 1965).

(1) Construction.

(a) Completed. 1905.

(b) Type. Rock mound (Fig. 34).

(c) Purpose. Protection of Venice Amusement Pier (since removed).

(2) Owner. Department of Tidelands and Submerged Lands, City of Los Angeles, California (as Trustee for State of California).

(3) Location (approximate). Santa Monica Bay off Venice, California.
Latitude: 33°59′N. – Longitude: 118°28′W.

(4) Physical Environment (Fig. 35).

(a) Protected Ocean Environment. Protected from north and east by proximity to shore; partially protected from west and south by Channel Islands and Santa Catalina and to southeast by Point Vincente.

(b) Wave Conditions. Sea and swell conditions fairly constant from south and southwest throughout year. Heaviest storm waves reportedly in 12- to 15-foot range, approach generally from northwest during winter months.

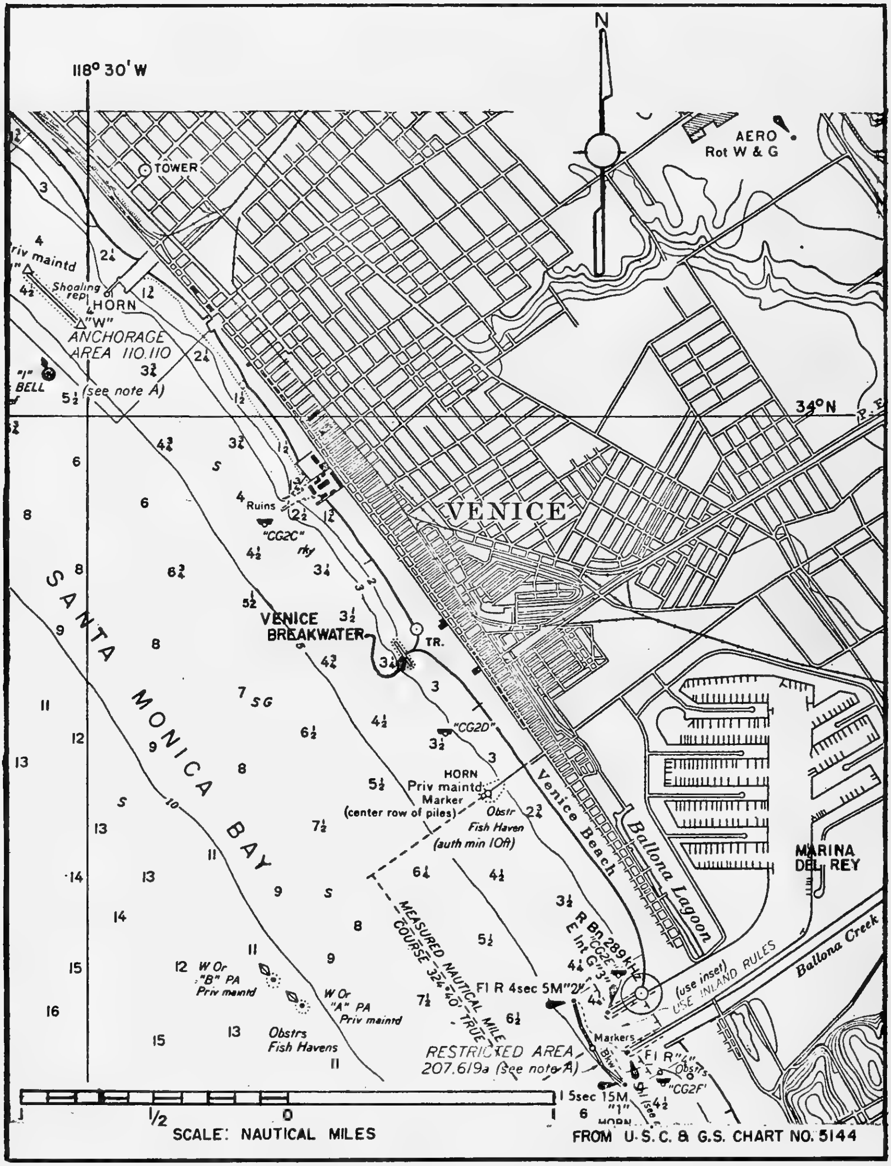

Figure 35. Location plan.

(c) Currents. Variable, depending mostly on wind; generally southerly in spring and summer, and northerly in fall and winter.

(d) Winds. Prevailing winds from west to west-southwest. Mean annual wind-speed 6 miles per hour with peak gusts recorded from the west at 62 miles per hour.

(e) Storm Surge and Tides.

Mean sea (tide) level, 2.8 feet above MLLW

Extreme tide level, 7.1 feet above MLLW

Mean tide range, 3.7 feet

diurnal, 5.4 feet, extreme, 10.0 feet

(f) Littoral Transport. Little since construction of Santa Monica Breakwater.

(g) Water Depth at Structure. 12 feet on offshore side.

(h) Bottom Conditions. Sandy.

(5) Structural Features. Engineering drawings were not available. Observations indicate the armor stone is granite, probably from Riverside, California quarries, about 175 pounds per cubic foot, used in graded sizes from 50 pounds to 10 to 11 tons each. No information on whether a different material was used for the core.

The slope on the seaward side is 1 on 1.5 or 1 on 2; steeper on the shore side. The breakwater is 600 feet long with a crest elevation about 12 feet above MLLW.

(6) Design Data. There is little available from reliable written records as to design data, designer's name, contractor's name, or construction costs. Built by a subdivider in the area, its original purpose was protection of the Venice Amusement Pier at Windward Avenue.

The builder, Abbott Kinney of Los Angeles, owned the breakwater until 1917, at which time the breakwater and the beach and pier, went through various legal changes in ownership. About 1948 or 1949 ownership was acquired by the City of Los Angeles in Trusteeship for the State of California.

(7) Structural Performance.

(a) Performance. Built as protection from offshore waves for the Venice Amusement Pier, the breakwater served its purpose well until the pier was demolished in 1948. Overtopped only by extreme waves.

(b) Maintenance. No maintenance work on the structure has been reported.

(8) Effect of Structure on Environment.

(a) Physical. This breakwater has provided a prime example of conditions which induce the natural formation of a tombolo, eventually connecting the detached breakwater to shore by accretion behind the breakwater (Fig. 36).

This breakwater, close to shore in comparison with its own length, with a wave shadow around the ends reflecting offshore, causes accretion from shore outward. Sounding surveys in 1935 and 1953 show the results of this action (Fig. 36). Local conditions have also contributed to the overall result.

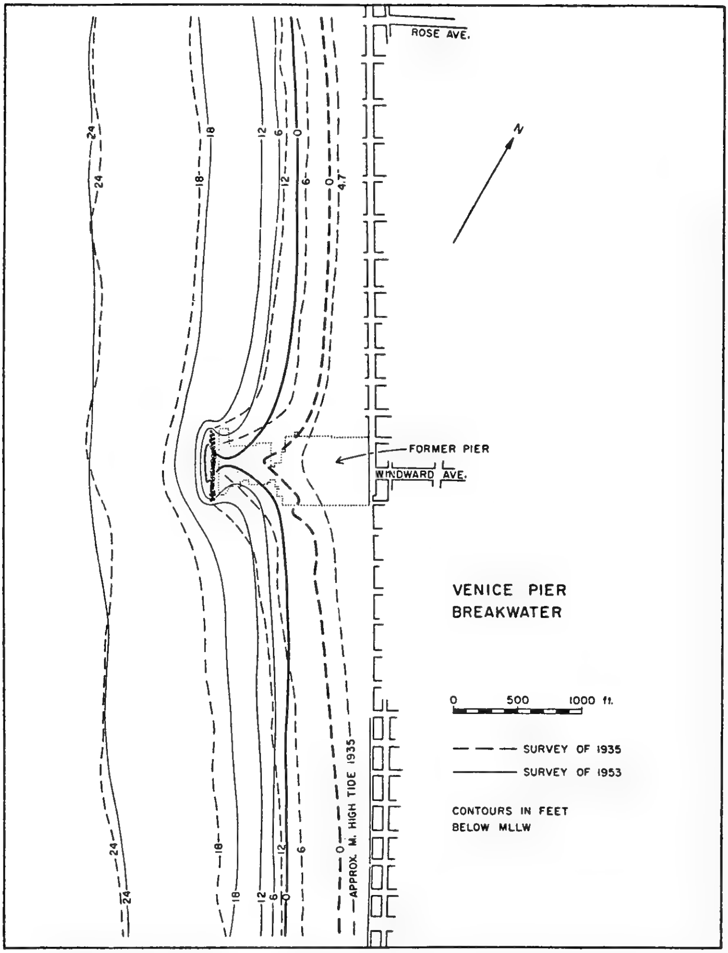

Figure 36. Venice Pier Breakwater showing the effect of a detached breakwater on adjacent beach. Comparative surveys by Los Angeles County (Inman and Frautschy, 1965).

In this area of southern California, serious accretion will take place if a structure is distant from shore less than three to six times its own length. In this case, the 600-foot-long breakwater, was originally located 1,000 feet offshore from the mean lower low water line.

The Venice Amusement Pier, with its piling and footings in the beach zone, contributed heavily to beach buildup and extension out to the breakwater in the immediate area.

Littoral transport in a southerly direction along the shore was 200,000 to 250,000 cubic yards per year, until construction in 1933 of the Santa Monica breakwater, approximately 1.5 miles up the coast. Stabilization began at this time and by the mid-1940's beach erosion due to littoral transport had stopped.

At the time the Amusement Pier was removed (1948), 14 million cubic yards of sand were deposited along the Venice Beach. By 1953, as shown on the sounding surveys, the tombolo had reached the breakwater and is still building.

(b) Biota. In the absence of recorded data, reports indicate heavy marine life in the breakwater area. Periodic pollution in the area is due to offshore seepage of natural asphalt and, previously, to solid sewage disposal at Los Angeles. However, these causes do not involve the structure in any way.

(c) Aesthetics. Probably because of the length of time in existence, there are no known aesthetic objections.

(9) Engineering. Unknown.

(10) Construction Contractors. Abbott Kinney, Los Angeles.

(11) Construction Cost. Unknown.

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U.S. DEPARTMENT OF COMMERCE, "United States Coast Pilot 7," 10*th* ed., 1968, p. 337.

d. Winthrop, Massachusetts.

Figure 37. View of breakwater from northeast, (1949) (U.S. Army, Corps of Engineers, Coastal Engineering Research Center, 1966).

(1) Construction.

(a) Completed. September 1935.

(b) Type. Rubble-mound breakwater (granite) (Fig. 37).

(c) Purpose. Shore protection and beach preservation.

(d) Auxiliary Purpose. Protected small-craft mooring area.

(2) Owner. Massachusetts Department of Public Works, 100 Nashua Street, Boston, Massachusetts 02114.

(3) Location. Atlantic Ocean in Broad Sound 1,000 feet off Winthrop Beach, 4.5 miles northeast of Boston, Massachusetts.

Latitude: 42°22′30″N. – Longitude: 70°57′58″W.

(4) Physical Environment.

(a) Protected Ocean Environment. Beach and breakwater are protected from westerly directions by mainland. Area is exposed to winds and ocean waves from northerly, through easterly to southerly directions.

(b) Wave Conditions. Exposed to wind-generated waves from northeast through east through southeast. Maximum waves in Boston area occur during December, January and February with a significant wave height of 23 feet exceeded on 0.1 percent of the days during those months (1 day in 1,000 days) (U.S. Naval Weather Service Command, 1970).

(c) Currents. Tidal currents in the area are reported as negligible.

(d) Winds. Winds from the northeast quadrant are prevailing and predominant (U.S. Congress, 1947).

(e) Storm Surge and Tides.

Extreme high tide, 15 feet above MLW

Extreme low tide, –3.0 feet below MLW

Mean normal tidal range, 9.0 feet

Mean spring tidal range, 10.4 feet

(f) Littoral Transport. Predominant direction of transport along Winthrop Beach is from north to south.

(g) Water Depth at Structure. Varies from 0 to 10 feet below MLW.

(h) Foundation Conditions. Available information indicates the breakwater rests on the base of eroded drumlins, hills of compacted clay, and sand and gravel with numerous cobbles and boulders.

(5) Structural Features (Fig. 38).

(a) Dimensions of Basic Structure.

1 Length. Five detached breakwater sections. One 400- and four 300-foot-long sections, spaced 100 feet apart at MHW. Overall length about 2,250 feet.

2 Side Slopes. Seaward slope, 1.75 to 1; shoreward slope, 1.5 to 1; and between sections, 2.0 to 1.

3 Crest Elevation and Width. +8.0 feet above MLW, 9.0 feet above MHW, and 12.0 feet wide.

(b) Unusual Structural Features. Gaps in structure provided to save stone, permit circulation and enable small boats to pass through. Gaps not wide enough to permit passage of sizeable wave. Cross-section shape in form of an ellipse to offer less resistance to overtopping wave.

(6) Design Data.

(a) Design Conditions. Not available.

(b) Model Study. None.

(c) Instrumentation. None.

(7) Structural Performance.

(a) Effectiveness of Structure. Good. Field surveys and observations indicate that breakwater has been effective in lessening wave action and in protecting the shore of Winthrop Beach for a distance equal to the breakwater length. Substantial accretion has taken place within this protected zone moving the shoreline seaward. Some erosion north and south of the protected area has occurred. Increased beach width has decreased wave attack on seawall protecting Winthrop, Massachusetts.

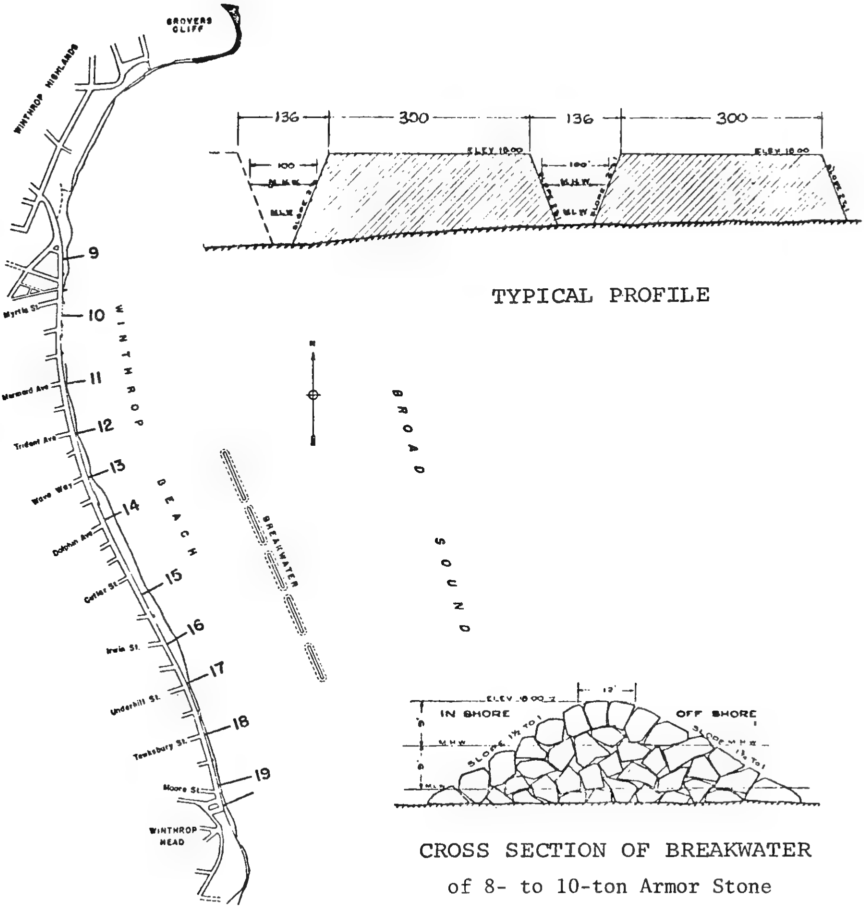

Figure 38. General plan and location of survey profiles. Comparative profiles shown in Figures 39 and 40 (Massachusetts Department of Public Works, 1933).

(b) Integrity of Structure. Good. During its 40-year lifetime, the breakwater has weathered numerous severe storms with little displacement of stone. Parts of the breakwater may have experienced slight subsidence but there is no observable damage. No maintenance has been performed.

(8) Effect of Structure on Environment.

(a) Physical. Since construction, the rate of volume change of the beach material has greatly increased. There has been accretion of beach material within the protected zone greater than the erosion which had occurred previously. Survey profiles shown in Figures 39 and 40 indicate that the buildup of material (profiles 14, 15 and 16) was provided from material eroded from the adjacent sections of the beach to the north and south (profiles 10, 11, 12, 18 and 19).

The erosion rate north of the breakwater increased after construction of the breakwater. Material from the beach north of the breakwater moves southward to the beach abreast of the breakwater but does not return to the north during periods of reversal in the direction of wave approach (as it did before construction of the breakwater).

At the south end of the breakwater, there has been a tendency for waves to work around the breakwater, over the bar and to wash out the beach just opposite its southerly end as shown in the beach profiles.

From 1938 through the 1950's as part of various protection and improvement plans, seawall modifications and rockfill groins were constructed and various quantities of sand were placed between the groins along the beach. Two areas were filled: north of the breakwater and at the south end of the beach behind the breakwater. The groins, built to retard the rate of erosion in these areas, have had limited success.

In 1971 more sand was brought in by truck and placed on the beach opposite the full length of the breakwater from the seawall towards the ocean. During a coastal storm in February 1972, a substantial amount was lost.

The structure is not a hazard to navigation, although small-craft operations have been restricted by shoaling near the breakwater. Recreational activities, bathing in particular, have benefited.

(b) Biota. In the absence of recorded data, oral reports indicate a general increase in marine life and bird population near the breakwater. Kelp and other vegetation have been noted on the stone at the structure toe.

(c) Aesthetics. The townspeople requested the breakwater rather than a higher seawall, for aesthetic reasons. No known aesthetic objections.

(9) Engineering. District Waterways Engineer, Massachusetts Department of Public Works, 100 Nashua Street, Boston, Massachusetts 02114.

(10) Construction Contractors (built in three stages).

(a) Stage 1, Merritt, Chapman and Scott Corporation, New York, New York 10001 (discontinued operations).

(b) Stages 2 and 3, William R. Farrell, Boston, Massachusetts 02109 (discontinued operations).

(11) Construction Dates.

(a) Stage 1, 27 June to 30 November 1933.

(b) Stage 2, 1 August 1934 to 6 January 1935.

(c) Stage 3, 9 July to 3 September 1935.

(12) Construction Cost.

(a) Stage 1 at $2.22 per ton,	$146,521.33
(b) Stage 2 at $1.97 per ton,	67,258.43
(c) Stage 3 at $2.17 per ton,	23,961.14
	$237,740.90

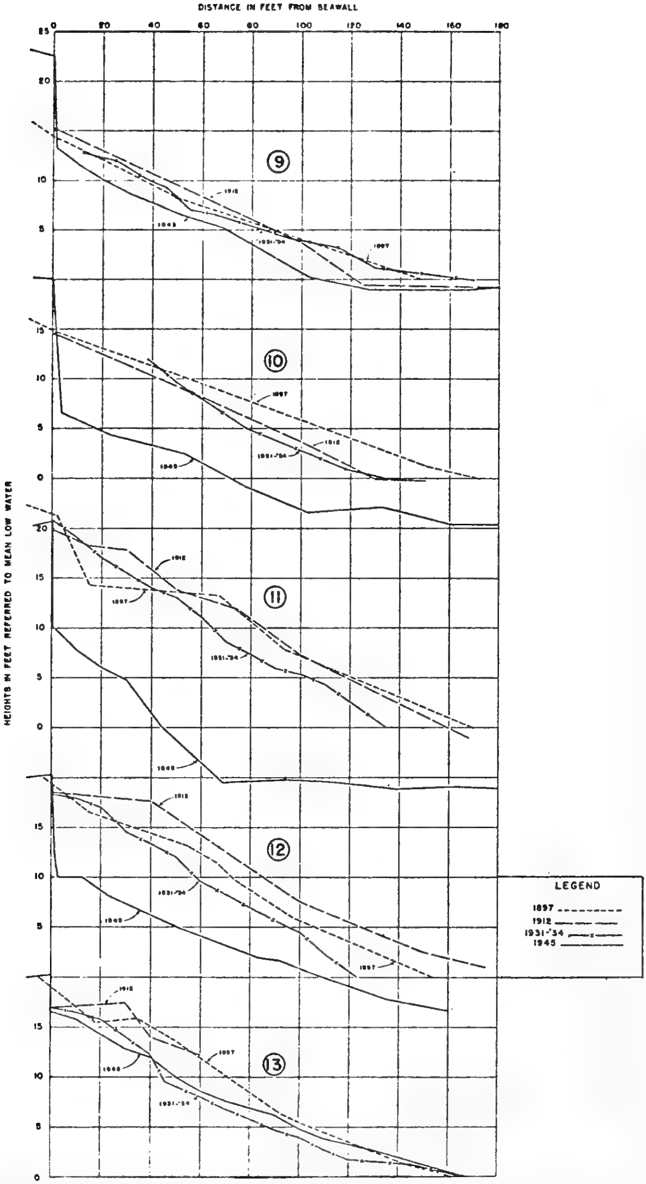

Figure 39. Comparative survey profiles 9 through 13.
Profile locations shown in Figure 38. (U.S. Congress, 1947).

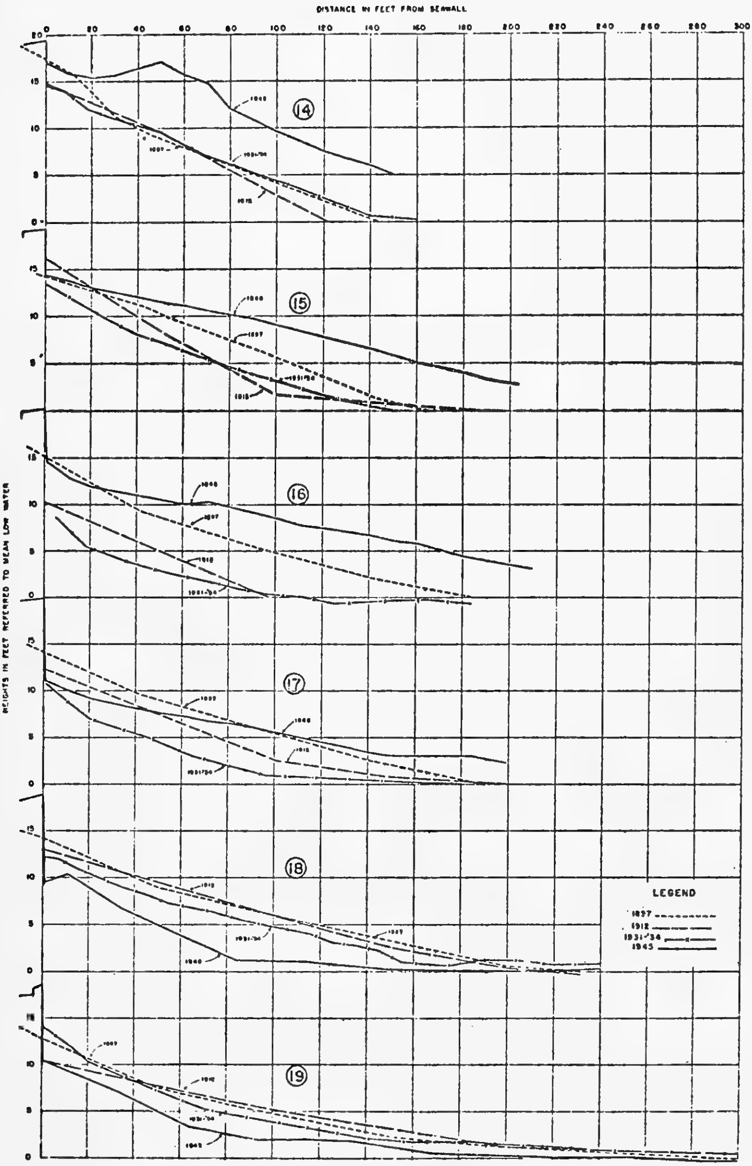

Figure 40. Comparative survey profiles 14 through 19. Profile locations shown in Figure 38. (U.S. Congress, 1947).

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U.S. ARMY ENGINEER DIVISION, NEW ENGLAND, "Beach Erosion Inspection Reports at Winthrop Beach, Mass," Waltham, Mass., Aug. 1959 to 1972.

4. Detached Breakwater—Middle and Long Beach Breakwaters—Long Beach, California.

Figure 41. View of west end of middle breakwater (August 1973).

a. Construction.

(1) Completed. 1949.

(2) Type. Sand and clay mound covered with rock blankets (Fig. 41).

(3) Purpose. Protection of Long Beach Harbor.

b. Owner. Department of the Army, Corps of Engineers, South Pacific Division, Los Angeles District, 300 N. Los Angeles Street, Los Angeles, California 90012.

c. Location (approximate). San Pedro Bay off Long Beach, California forming southern boundary of harbor.

Latitude: 33°43′N. – Longitude: 118°12′W.

d. Physical Environment (Fig. 42).

(1) Protected Ocean Environment. Ocean environment is protected through the northern half by proximity to shore; partially protected from south and southwest by Catalina and the Channel Islands, but fully exposed to southeast.

(2) Wave Conditions. Seas and swells are predominantly from the northwest (U.S. Navy Hydrographic Office, 1969), but reaches a 20-foot range on the ocean side during southeast storms.

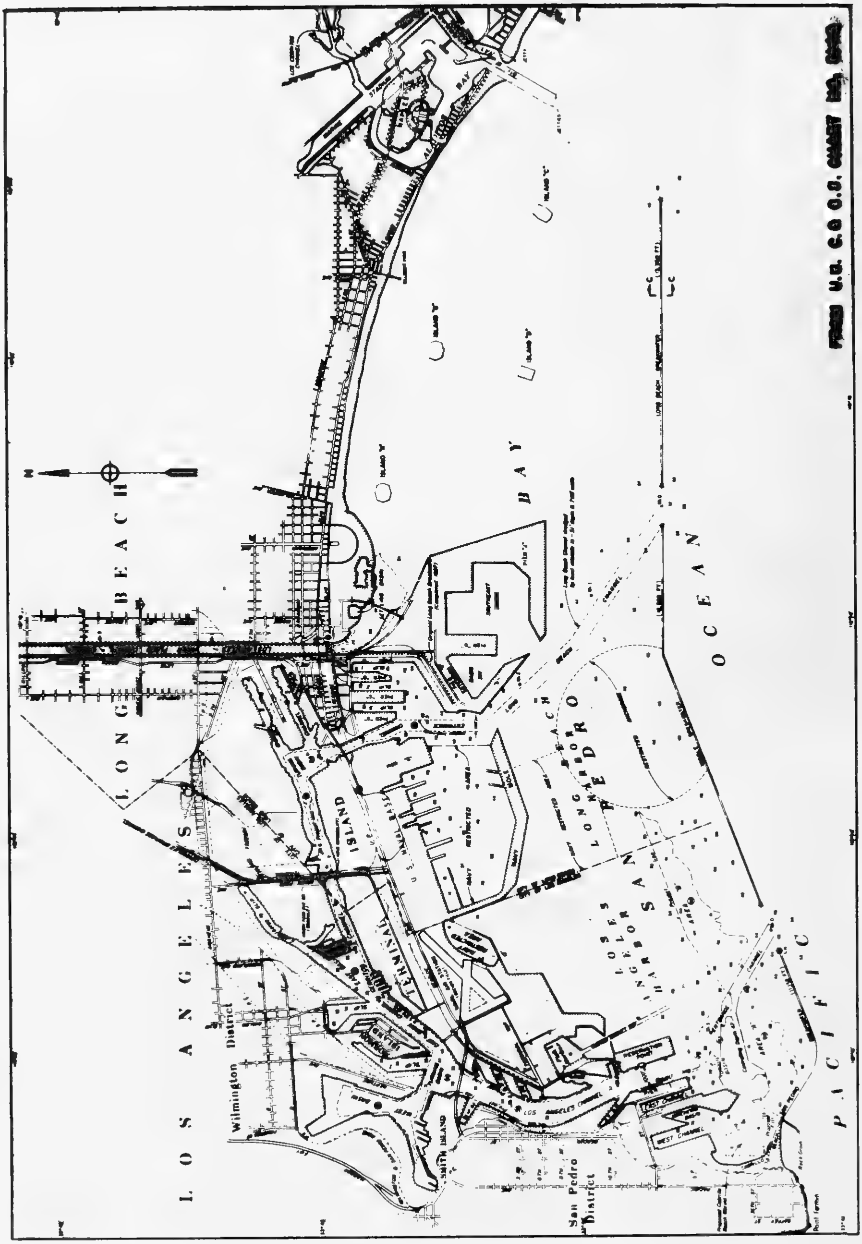

Figure 42. Location plan.

(3) Currents. Largely northwest from July through February, and southeast from March through June (U.S. Navy Hydrographic Office, 1947).

(4) Winds. Prevailing west-southwest winds from April through September; west winds from October through March, with maximum recorded velocity of 54 knots during this period (Department of Commerce, 1968).

(5) Storm Surge and Tides (U.S. War Department, U.S. Engineer Office, 1939).

MHHW, 5.4 feet above MLLW

Mean tide level, 2.8 feet above MLLW

Extreme low, 2.5 feet below MLLW

Mean tide range, 4.0 feet

Extreme tidal range, 10.0 feet

(6) Littoral Transport. Negligible.

(7) Water Depth at Structure. Built generally along a 50-foot depth contour with actual depths varying from 46 to 56 feet at MLLW.

(8) Foundation Conditions. Generally hard *shaly* clay.

e. Structural Features (Figs. 43 and 44).

(1) Dimensions of Basic Structures. Length, 12,500 feet at 55° bearing; middle breakwater, 6,000 feet at 90° bearing; total, 18,500 feet; Long Beach breakwater, 13,350 feet at 90° bearing.

Side slopes:

Class A capstone: 1.5 to 1 (harbor side), 2.0 to 1 (ocean side)

Class B capstone: 1.25 to 1 (harbor side), 2.0 to 1 (ocean side)

Crest elevation and width: 14 feet above MLLW, 16 feet wide

(2) Unusual Structural Features. Designers, taking advantage of exceptional type of stiff *shaly* clay available from channel and harbor dredging in the area, substituted this material for rock in the lower levels on a yard-for-yard basis, to keep construction costs to an absolute minimum. The clay is not plastic, but a very sandy incipient shale of high colloidal content, and is not subject to flow except under great pressure. Stones of Classes A, B, and later C were specified according to maximum and minimum size and gradation (Marcy, 1935). The B stones were dumped by barge while the A-stone caprocks were carefully placed by derrick to secure better interlocking, reduce voids and provide smoothness. Dropping of stones on earth layers also aided compaction and settlement.

f. Design Data.

(1) Design Conditions:

Depth at structure, 46 to 56 feet at MLLW

Extreme tide range, 10 feet above MLLW

Maximum depth, 66 feet

Design wave, 27 feet (from September 1939 storm conditions)

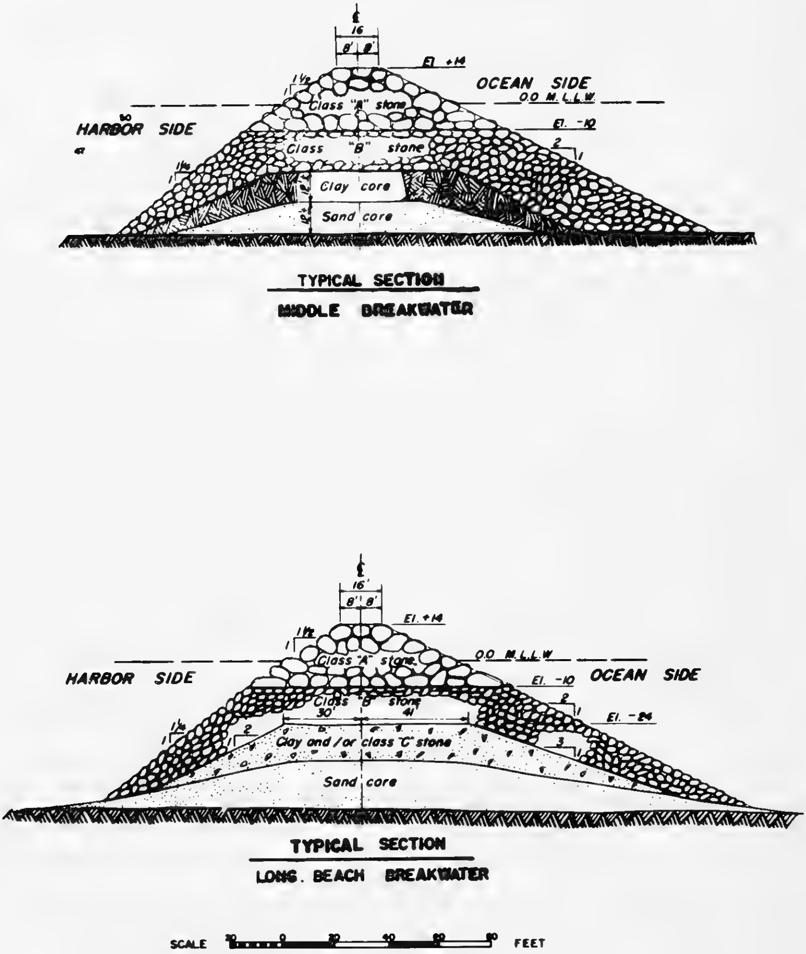

Figure 43. Typical sections of breakwaters.

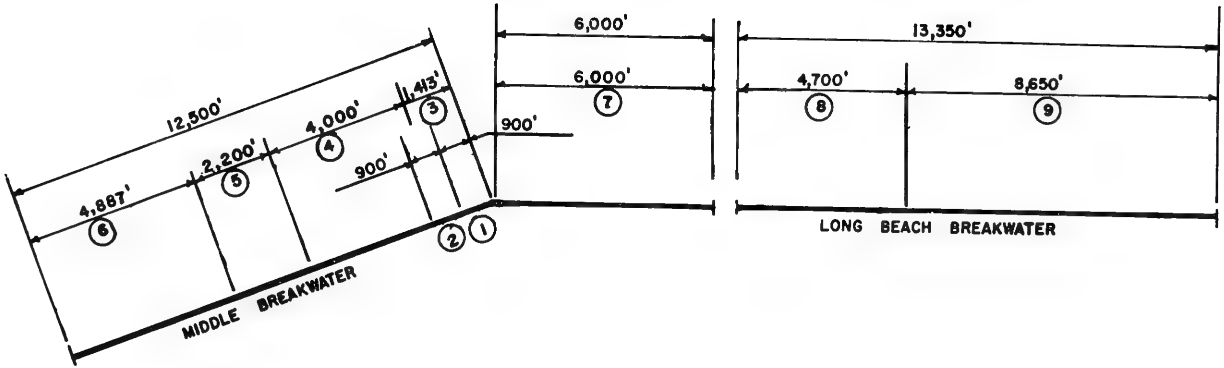

Figure 44. General plan of breakwaters. Circle numbers indicate breakwater sections covered by various construction contracts mentioned in report.

(2) Testing. Extensive tests were made on the clay before incorporating in design. It was found that no flow occurred until the pressure reached 6,500 psi (Marcy, 1935).

(3) Model Study. None. A model study is underway at U.S. Army Engineer Waterways Experiment Station Testing Laboratory (WES), Vicksburg, Mississippi, in connection with increasing the harbor capacity within the breakwaters.

(4) Instrumentation. None.

g. Structural Performance.

(1) Performance. No indication of failure, although a single capstone in various locations has been upended due to wave action. In 1968, at one point near the eastern end of the middle breakwater, a ship struck the breakwater from the ocean side, dislodging several stones (Fig. 45). Since no further damage was apparent, no repair has been undertaken. Settlement due to breakwater construction has been negligible.

Figure 45. View on ocean side of middle breakwater showing displaced capstones (August 1973).

(2) Maintenance. None to date.

h. Effect of Structure on Environment.

(1) Physical. No appreciable accretion or scour in breakwater area. Harbor and channel remain at dredged depths with little maintenance dredging.

(2) Biota. Provides haven for marine life including birds and seals. Considerable sport fishing in the immediate area.

(3) Aesthetics. Breakwater is far enough seaward to affect the view from shore.

i. Engineering. U.S. Army Engineer District, Los Angeles, California.

j. Construction Contractors (Nine sections) (Fig. 44).

Section	
1, 2	Standard Dredging Company
3	Puget Sound Bridge and Dredging Company
4, 5, 6	Rohl-Connolly Company
7, 8	Contractor's name not available
9	Connolly-Cass-Kiewits

k. Construction Dates.

Section	
1	April to July 1932 (experimental mound with proportions of sand to clay being varied, height from 22 to 35 feet above MLLW).
2	October 1932 to January 1933 (sand and clay only).
3	March 1933 to July 1934 (enrockment).
4	November 1933 to May 1935 (dredging, core construction and enrockment).
5	August 1935 to July 1936 (dredging, core construction and enrockment).
6	August 1936 to December 1937 (dredging, core construction and enrockment).
7	Dates not available.
8	Completed December 1941.
9	June 1946 to 1949.

l. Construction Costs (Berry, 1946; U.S. Army, Corps of Engineers, 1939).

Section	
1	$ 50,536.61
2	70,862.77
3	584,838.12
4	1,595,367.82
5	996,903.67
6	2,346,302.67
7, 8	.... amount not available
9	$ 7,447,700.00
Total	$25,000,000.00 (approximate)

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III. PILE-SUPPORTED STRUCTURES

1. Diamond Shoals Light Station—Cape Hatteras, North Carolina.

Figure 46. Aerial view of Diamond Shoals Light Station.

a. Construction.

(1) Completed. October 1966.

(2) Type. Pile-supported jacket-type structure (Fig. 46). Steel piles driven through a prefabricated steel-trussed template. Superstructure of prefabricated sections.

(3) Purpose. Aid to navigation (replace lightship).

b. Owner. U.S. Coast Guard, Fifth Coast Guard District, 431 Crawford Street, Portsmouth, Virginia 23704.

c. Location. Atlantic Ocean about 13 miles east-southeast of Cape Hatteras, North Carolina.

Latitude: 35°09′12″N. – Longitude: 75°17′48″W.

d. Physical Environment.

(1) Open Ocean Environment. Historically known as one of the most severe weather areas of the U.S. east coast. Structure is exposed to ocean winds and waves from all directions. Waves from northwest are generally smaller because of shoals and limited ocean fetch.

(2) Wave Conditions. Predominant sea conditions vary in direction and intensity seasonally with prevailing winds. Deepwater waves from northeast during northeaster storms and from the southeast through southwest during hurricanes present the most adverse sea conditions. Waves higher than 40 feet have been recorded.

(3) Currents. Currents in the area are influenced by the Gulf Stream, a prevailing northeast flow off the coast throughout the year. Surface currents at the site have been reported in excess of 4 knots.

(4) Winds. The wind at Cape Hatteras averages 8 to 11 knots year round. Prevailing winds are southerly during April through August and northerly September through March. Maximum recorded windspeed is 110 miles per hour.

(5) Storm Surge and Tides.

Extreme high tide (approximate), 7.6 feet above MLW

Extreme low tide (approximate), 2.0 feet below MLW

Mean normal tidal range, 3.4 feet

Mean spring tidal range, 4.1 feet

(6) Littoral Transport. Not applicable (structure far from shore).

(7) Water Depth at Structure. 54 feet below MLW.

(8) Foundation Conditions. Structure is located on a sand ridge. Borings at site reveal that soils are basically granular consisting of fine to medium sand approximately to elevation –122 feet underlain by silty sand with some clay seams and pockets to elevation –220 feet. Shell fragments are prevalent throughout the sands.

e. Basic Structural Features (Fig. 47).

(1) Dimensions of Basic Structure (U.S. Coast Guard, 1964).

(a) Steel Jacket Structure. 80 feet high by 70 feet square at base with 39-inch diameter by 0.75-inch wall-pipe legs at each corner battered 1:12 to outside. Legs serve as sleeves for pipe piles. Framing members 18 inches in diameter.

(b) Steel Pipe Piles. 33-inch diameter about 225 feet long (1.12-inch wall from elevation +15 to –100 feet and 0.75-inch wall from elevation –100 feet to –210 feet).

(c) Steel Deck Section. 56 feet high by 56 feet square at top with 33-inch diameter by 1.12-inch wall pipe at corners. Framing members 14 and 18 inches in diameter.

(d) Machinery and Quarters Deck. 70 by 70 feet.

(e) Deck Elevations (referred to MLW). Boat landing, +11.8 feet; maintenance, +61.0 (10 feet above maximum wave elevation); quarters, +79.0; helicopter, +90.3; and lantern platform, +117.4 feet.

(2) Unusual Structural Features. Template-type structure developed from successful use in Gulf of Mexico and adapted to open Atlantic Ocean. Components of structure were prefabricated onshore to simplify erection at sea. Tower consists of a template structure fixed to the bottom by pipe piles driven through the legs, surmounted by a deck section welded to these piles, which supports the storage tanks and deckhouse.

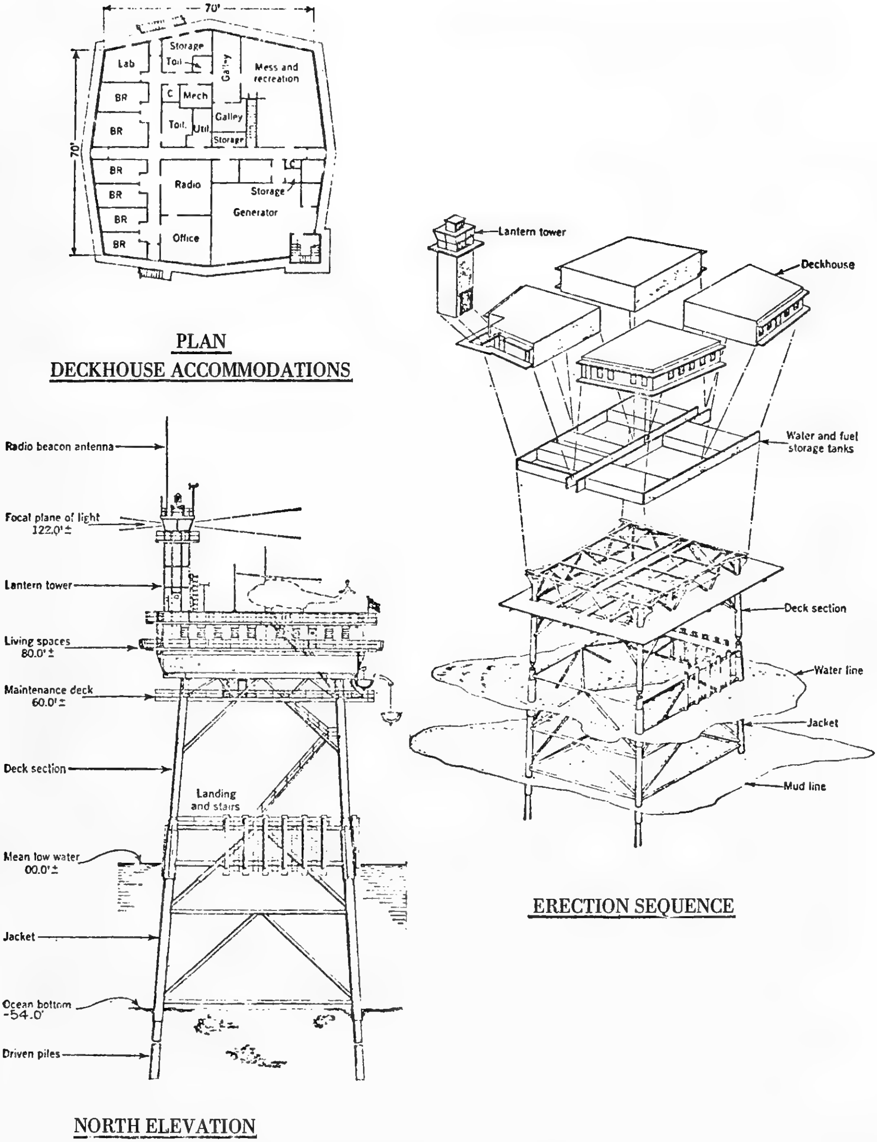

Figure 47. Basic structural features.

f. Design Data (U.S. Coast Guard, 1963). (Conservative design because of remoteness from shore.)

(1) Operational Tenure. 75 years.

(2) Maximum Design Storm Tide. 14.0 feet above MLW.

(3) Design Wave. Maximum height 52 feet.

(4) Depth at Structure. 54 feet below MLW.

(5) Design Windspeed. 125 miles per hour.

(6) Instrumentation. Structure originally provided with instrumentation for obtaining meteorological and oceanographic data and also a device for oceanographic sampling of waves, water levels, salinity, and temperature (sampling device no longer operable). It is reported that the tower is to be reinstrumented for obtaining sea temperature, current direction and velocity, sand bottom movement, and wave measurements thus providing a good source of environmental reference data.

g. Structural Performance (Rouzie, 1967–71).

(1) Effectiveness of Structure. Excellent. Structure is performing satisfactorily. Design conditions have not been experienced. The davits and lifeboat have been removed. A general purpose hoist for supplies and personnel has been added.

(2) Integrity of Structure. Good. Primary components of the tower are in excellent structural condition. Anticipated cleaning and painting has been performed. Glass of the lantern tower (elevation 122 feet) was broken by a waterspout and repaired. Ultrasonic weld inspection and general inspection of the structure above and below water (June 1970) confirmed competency of the structure but revealed continued scouring around the tower legs, averaging 3 feet in depth (Rouzie, 1967–71). Cathodic protection sacrificial anodes were replaced in 1970.

h. Effect of Structure on Environment.

(1) Physical (Rouzie, 1967–71). The offshore distance of the tower precludes any effect on the shoreline. Local scouring of the sea bottom at the base of the structure is monitored by measurements taken during periodic underwater inspections. Measurements in 1970 indicated a maximum scour of 11 feet at the legs. A year later, with no remedial action taken, this had decreased to a maximum of 5 feet. Graded riprap or concrete scour protection, originally recommended but not installed, is being considered as a possible solution to the problem. Structure psychologically beneficial to the small-craft operator.

(2) Biota. No recorded data. Several underwater inspections have reported heavy marine growth on the structure and that fish are attracted to the structure (Fig. 48). Occupants report birds are also attracted.

(3) Aesthetics. Light station is not visible from shore, thus the offshore view is not aesthetically affected. No known objections from occupants of passing boats.

i. Engineering. U.S. Coast Guard Headquarters, Washington, D.C.

j. Construction Contractors (joint venture). Tidewater Construction Corporation (sponsor), P.O. Box 57, Norfolk, Virginia 23601; Raymond International Incorporated, 2801 South Post Oak Road, Houston, Texas 77027; and Peter Kiewit Sons' Company, 1000 Kiewit Plaza, Omaha, Nebraska 68131.

k. Construction Date. Construction period from 14 March to 24 October 1966.

l. Construction Cost. Contract value, $2,025,000.

Figure 48. View showing marine life beneath tower.

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2. Offshore Production and Gathering Facilities–Eugene Island Area, Louisiana.

Figure 49. Aerial view of Eugene Island area facilities.

a. Construction.

(1) Completed. August 1955.

(2) Type. Concrete platform supported on concrete piles driven with concrete template (Fig. 49).

(3) Purpose. Support of offshore oil production and gathering facilities.

b. Owner. Magnolia Petroleum Company (subsidiary of Mobil Oil Company), Dallas, Texas 75221.

c. Location (approximate). Eugene Island Area, Block 126, offshore from Louisiana Gulf Coast, generally south-southwest of Morgan City, Louisiana.

Latitude: 29°00′N. – Longitude: 91°31′W.

d. Physical Environment (Fig. 50).

(1) Open Sea Environment (Gulf of Mexico). Distance from shore leaves structures fully exposed in all directions.

(2) Wave Conditions. 32-foot-maximum wave (breaking).

(3) Currents. Not significant.

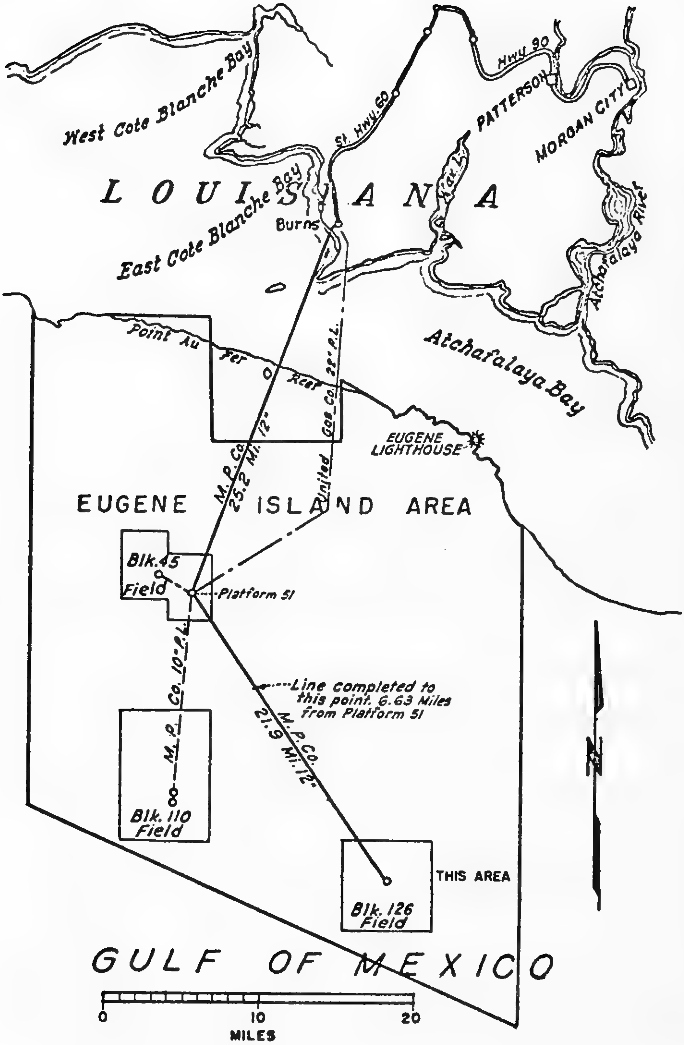


Figure 50. Location plan.

(4) Winds. 150 miles per hour (hurricane season in late summer and fall).

(5) Storm Surge and Astronomical Tide. 9 feet total.

(6) Littoral Transport. None; offshore location.

(7) Water Depth at Structure. 37 feet at MLW.

(8) Foundation Conditions. Piles driven through soft silty clay bottom into medium dense sand layer with shell fragments.

e. *Basic Structural Features* (Figs. 51 through 54).

(1) Dimensions of Basic Structures.

Oil storage platform, 50 by 100 feet
Separator platform, 98 by 136 feet
Pump and compressor platform, 72 by 141 feet
Living quarters platform, 44 by 87 feet

(2) Unusual Structural Features. All structures except living quarters platform constructed with Raymond concrete cylinder-pile template (rather than usual steel template structure) with 36-inch-diameter concrete cylinder piles driven through template supporting concrete platform decks. Living quarters platform is a jack-up rig with barge-type deck structure supported on 72-inch-diameter steel caissons.

All concrete design selected for savings in corrosion protection and maintenance costs.

f. *Design Data* (Figs. 52 and 53).

(1) Design Conditions.

Depth at structure, 37 feet at MLW
Astronomical tide, 2 feet
Storm surge, 7 feet
Maximum depth, 46 feet
Design wave height, 32 feet
Wind load, 150-mile per hour hurricane winds

(2) Model Study. None made.

(3) Instrumentation. None.

g. *Structural Performance.*

(1) Performance. Structure's performance has been very good, particularly since the full design conditions have occurred during two hurricanes. Although personnel are evacuated during such storms, winds are recorded and, from evidence of damage, it is possible to estimate the height reached by the seas during storms. Winds of about 150 miles per hour have been recorded and damage to walkways has indicated that waves have reached a height of 40 feet above MLW. Sand has been deposited in the living quarters a few feet above that. With personnel still on the platform, waves cresting at 15 feet above MLW have been observed. Normal seas crest at approximately 5 feet above MLW.

It should be noted that, while the owner's design practice to establish the physical environment has not changed, his method for computing wave forces has changed so that the design is below his present standards.

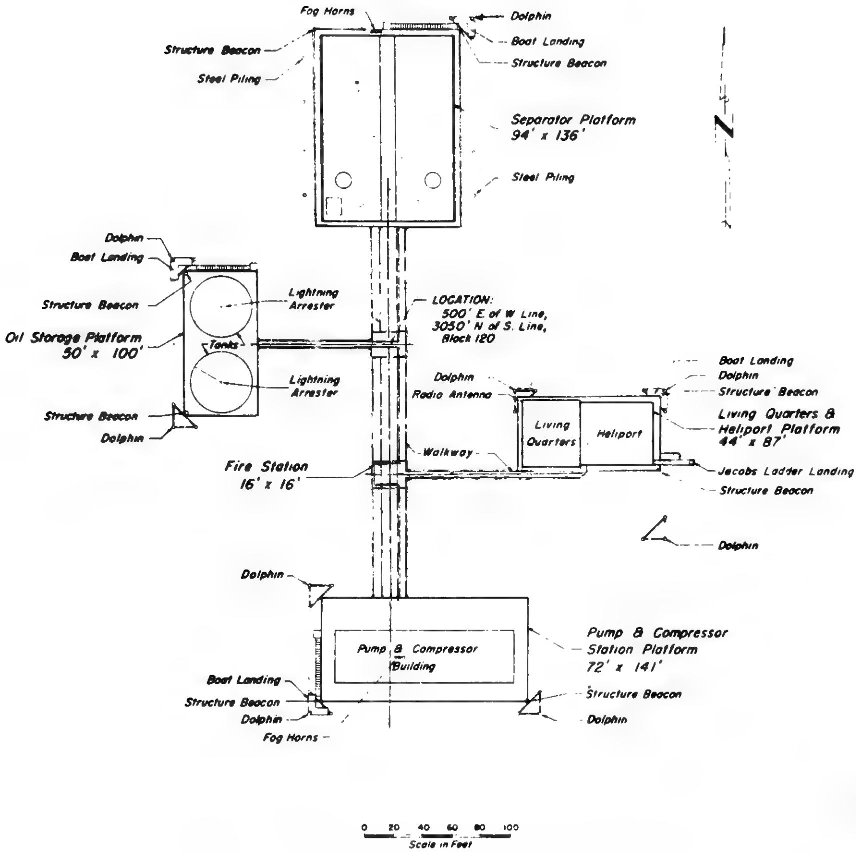


Figure 51. Proposed layout of platforms and walkway.

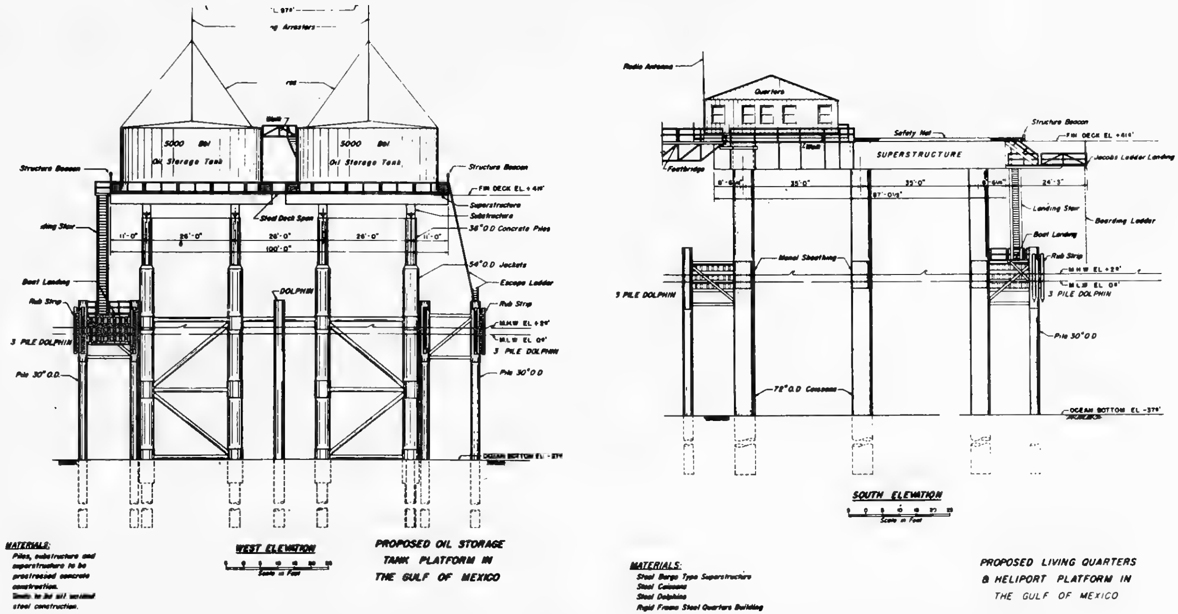
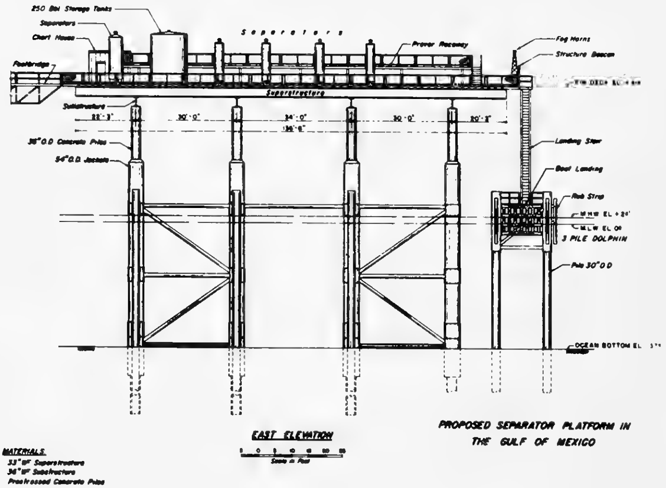


Figure 52. Proposed oil storage tank platform, heliport platform and living quarters.



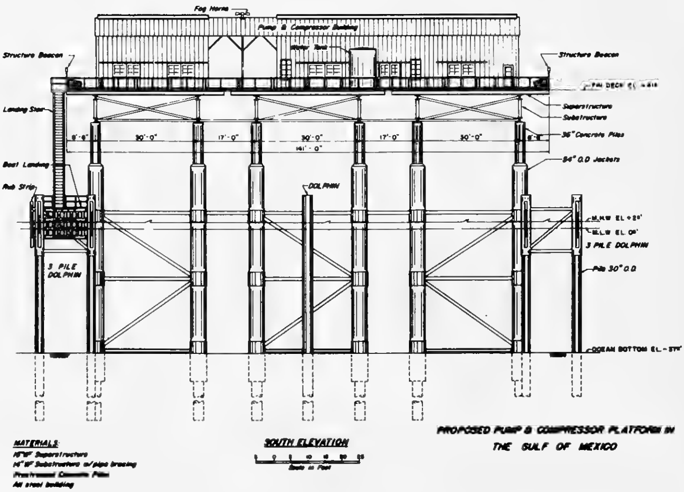


Figure 53. Proposed separator, pump, and compressor platforms.

Figure 54. View of oil storage platform showing concrete jacket, piles, and deck (upper photo); bottom views show the concrete jackets and piles, with gunited steel pipe bracing.

One support suffered a head-on collision by a work boat which broke through both jacket and pile wall, leaving a hole 5- by 1.5-feet wide. This was repaired by enlarging the break to sound concrete, wrapping a steel sleeve around the section, and grouting the broken area and the interior.

(2) Maintenance. Regularly scheduled inspections are made of the structural areas of the platforms, including a divers' inspection of the underwater parts. The only deterioration noted has been the spalling off of the Gunite coating on the steel collars and cross braces (Fig. 54). All piling and bracing between water surface and platform are regularly sandblasted and painted. This has been satisfactory in preventing corrosion of the bare steel, but the owner feels that using galvanized steel would be advisable in the future.

h. Effect of Structures on Environment.

(1) Physical. Scour attributable to storms and surface waves exists at all structures with the depth continually changing. Scour has reached a maximum known depth of 8 feet, but averages 3 feet. However, no records have been maintained.

(2) Biota. Some sea growth has developed on the concrete surfaces but more on the steel. Fish are plentiful in the area, but concentrations vary daily because of the numerous platforms. The Louisiana Wildlife and Fishery Commission is of the opinion that the total fish population has not changed with erection of the offshore structures as evidenced by the consistency of total tonnage taken annually by commercial fishermen.

(3) Aesthetics. Due to location 25 miles offshore and in an area devoted completely to the oil industry, aesthetics have had little influence on construction.

i. Engineering (three concrete platform structures only). Magnolia Petroleum Company (subsidiary of Mobil Oil Company), Dallas, Texas 75221; and Raymond Concrete Pile Company, New York, New York 10001.

j. Construction Contractors.

(1) Onshore Prefabrication. Gulf Prestressed Concrete Products, Morgan City, Louisiana 70380.

(2) Offshore Erection. J. Ray McDermott, Incorporated, Harvey, Louisiana 70058.

k. Construction Date. November 1954 to August 1955.

l. Construction Cost. Contract value: $1,500,000 (three concrete platform structures only).

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3. Drilling-Production Platforms.

a. "Monopod" Platform—Cook Inlet, Alaska.

Figure 55. Aerial view of "monopod" platform.

(1) Construction.

(a) Completed. October 1966.

(b) Type. Steel center post with flooded pontoon base supporting operations platform (Fig. 55).

(c) Purpose. Offshore oil drilling and production operations.

(2) Owners. Union Oil Company, 461 Boylston Street, Los Angeles, California 90017, and Marathon Oil Company, Findlay, Ohio 45840.

(3) Location (approximate). Cook Inlet, Alaska, Trading Bay oil field in Middle Ground Shoal area of Upper Inlet, 60 miles southwest of Anchorage.

Latitude: 60°50′N. — Longitude: 151°35′W.

(4) Physical Environment (Fig. 56).

(a) Protected Bay Environment. Location off Gulf of Alaska places site in area of some of the most severe weather and sea conditions in the world. These conditions have a strong influence at the site, which are also affected by the surrounding steep mountain terrain (Petroleum Engineer, 1971).

(b) Wave Conditions. 28-foot waves.

(c) Currents. Tidal currents up to ±8 knots at entrance to Upper Inlet, parallel to shoreline.

(d) Winds. In early summer, fresh northwesterly; in summer, easterly; in late summer, southwesterly; mean speed 6 knots (U.S. Coast and Geodetic Survey, 1964), maximum speed up to 66 knots (Petroleum Engineer, 1968).

(e) Storm Surge and Tides. Tide range, 25 to 30 feet.

(f) Littoral Transport. Alternating tidal action carrying glacial silt, sometimes so thick as to appear as liquefied mud.

(g) Water Depth at Structure. 62 feet at mean low tide (Cloyd, 1968).

(h) Foundation Conditions. Flat stiff clay bottom, strewn with boulders up to 30 feet in diameter (Visser, 1969; U.S. Coast and Geodetic Survey, 1964); subject to earthquakes.

(i) Ice Conditions. Average freezeup, 10 December; average breakup, 2 April; maximum thickness, 6 feet (approximate).

(j) Temperature. Water, 29° to 55°F (Blumberg and Strader, 1969); air, –40°F.

(k) Earthquake. Maximum recorded, 8.7 on Richter Scale in 1964.

(5) Basic Structural Features (Figs. 57 and 58).

(a) Dimensions of Basic Structure (Cloyd, 1968).

1 Platform. Two working decks, 110 by 110 feet; upper platform 19 feet above lower.

2 Support Column. 28.5-foot diameter by 125 feet high with 1-inch plate end sections, and 2-inch plate in middle section.

3 Pontoon Base. Two parallel pontoons, 20- to 24-foot diameter by 174 feet long, spaced at 120 feet, each secured to bottom by 8- to 36-inch diameter by 100-foot piles at each end.

4 Weight. 2,800 tons (ready to launch); 5,000 tons (approximate total).

(b) Unusual Structural Features. Design and construction using high strength steel to meet requirements of extreme site conditions of low temperatures, strong tidal currents, and pack ice. Steels used were Armco Lo Temp (A537 Grade A) and Super Lo Temp (A537 Grade B). Designed buoyant, so that the structure could be towed to Cook Inlet, and the base flooded for sinking. After anchoring by piles, the pontoons serve as oil storage tanks. After positioning, jackets having provided buoyancy are filled with concrete to a point above highest anticipated ice-level for protection.

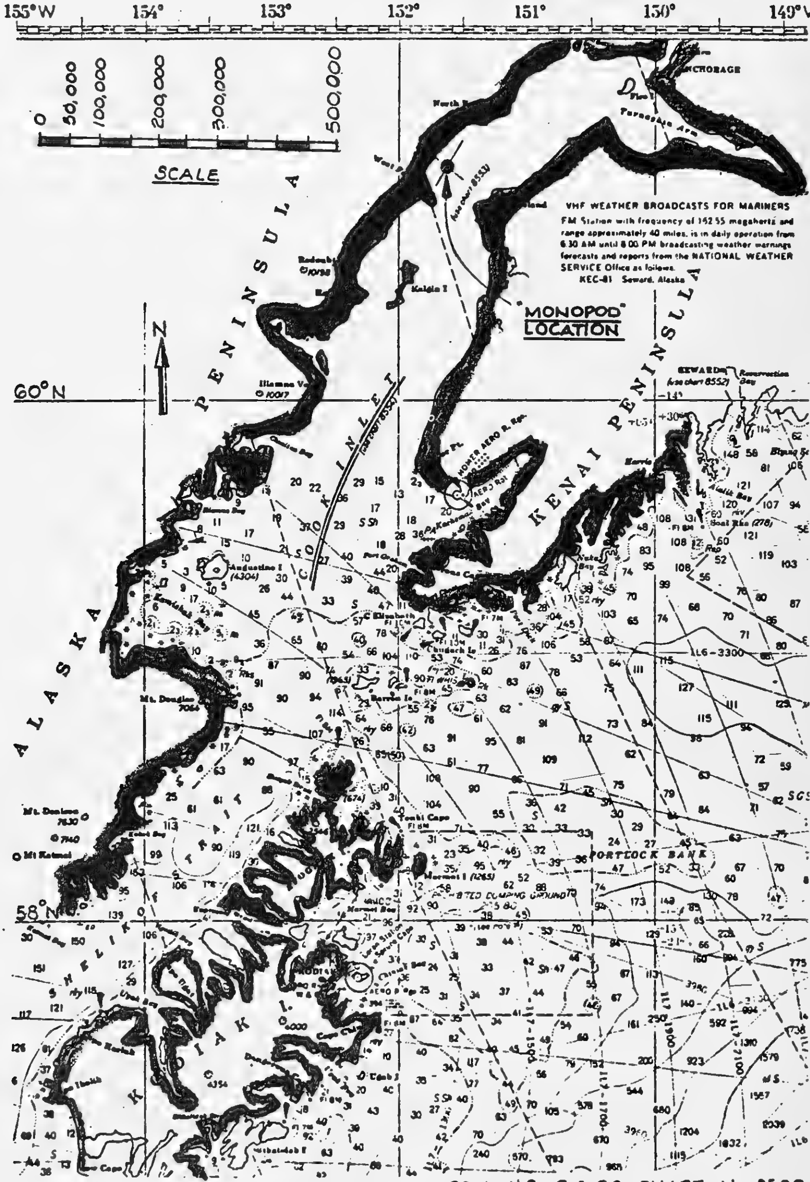

Figure 56. Location plan.

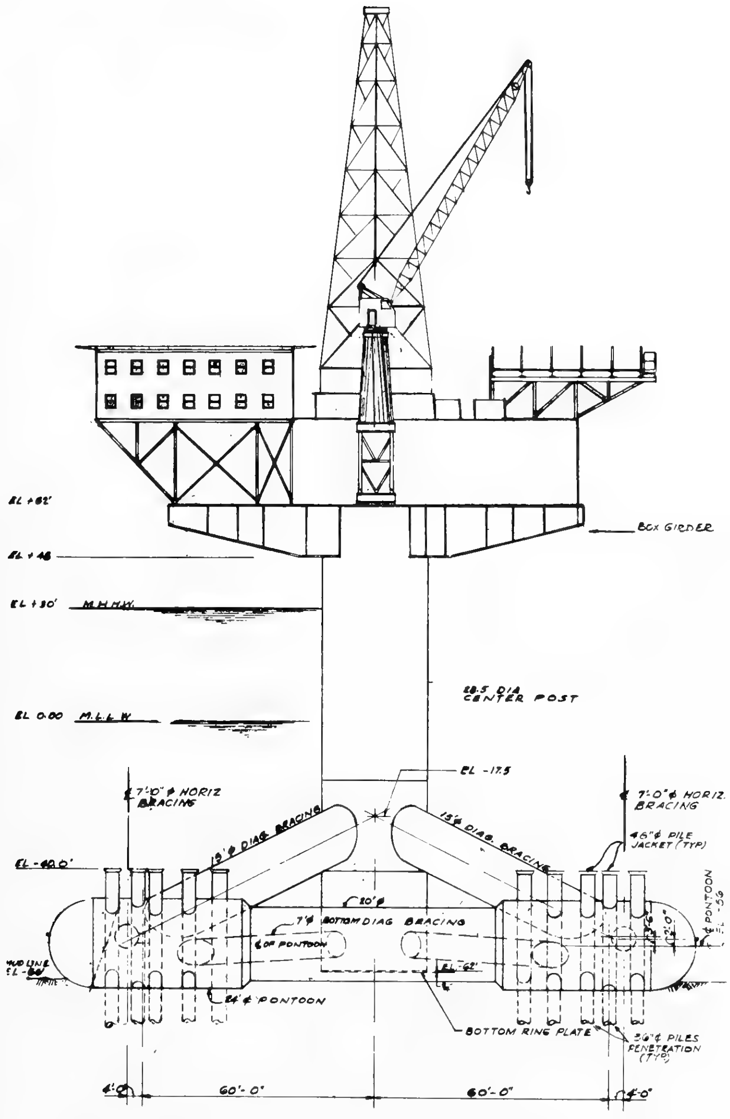

Figure 57. Structural features—elevation A.

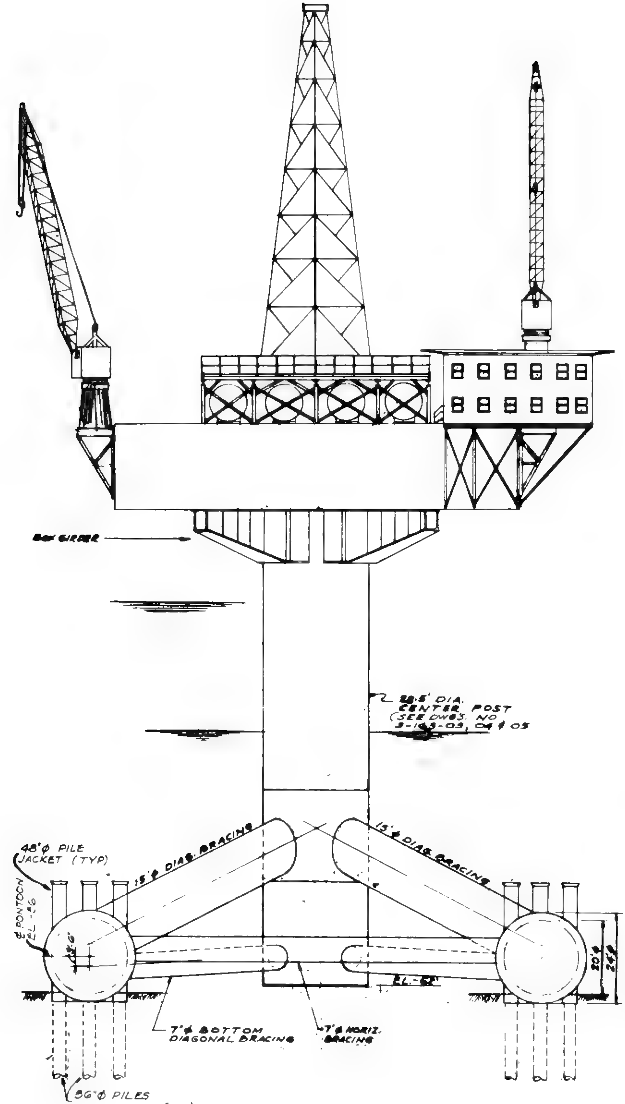

Figure 58. Structural features—elevation B.

(6) Design Data (Blumberg and Strader, 1969; Cloyd, 1968).

(a) Design Conditions.

Depth at structure, 62 feet at mean low tide.

Tidal range, 30 feet at mean low tide.

Maximum depth, 92 feet at mean low tide.

Design wave, 28 feet at 8.5-second period exerting: 600 pounds per square foot at crest, and 200 pounds per square foot at bay bottom.

Current loading, 120 pounds per square foot on flat members, and 40 pounds per square foot on round members.

Wind, 40 pounds per square foot on flat members, and 24 pounds per square foot on round members (based on 65 miles per hour with gusts to 100 miles per hour.

Earthquake-loading mass coefficient, 0.10 to 0.15 (depending on deck loading).

Water temperature range, 28° to 46°.

Ice-crushing strength, 300 pounds per square foot at 2 feet below water surface (based on temperature, thickness, and rate of loading).

Ice movement from any direction was the governing load, occurring during ice breakup with ice *pans* up to 10-foot diameter moving back and forth with current.

(b) Model Study (Cloyd, 1968). Two models of the monopod jacket were constructed. The smaller one was used to evaluate towing requirements and stability under tow. The tests indicated that the jacket in its towing configuration would have a draft of about 13 feet with its centers of buoyancy and gravity 8 and 47 feet above the jacket's low point.

The second model, larger and more complex, was constructed to study the jacket characteristics during the critical period of setting. As a result the structure was lowered into one end of the pontoons first and then settled level onto the other end rather than sunk balanced and erect all the way as originally planned. To take advantage of the best possible sea conditions, the structure was set during a 45-minute slack tide using its own individually controlled anchor system.

Diving operations were hampered by fast currents, zero visibility, and low temperatures; and floating equipment by the strength of the currents and the directional variation.

(c) Instrumentation (Geminder, 1968). Underwater strain-gages were placed to determine stresses and strains, to calculate ice force on the structure, and to measure load distribution within the structure. Data on ice forces (the governing factor in the design) are important to evaluate the present criteria and to set criteria for future work.

(7) Structural Performance (Fig. 59).

Figure 59. View showing effect of surface currents around "monopod" post (August 1973).

(a) Performance. Structural performance has been good, though design conditions have not been experienced. During periods of vibration, motion has been slow and regular; motion on nearby four-legged platforms has been more rapid and irregular.

The impressed current system of cathodic protection was not satisfactory, and is being modified and sacrificial anodes included.

A 0.5-inch double plate has been installed around the center post in the tidal area for erosion protection.

(b) Maintenance. Regularly scheduled inspections, including divers' inspection, has kept maintenance work to a minimum.

(8) Effect of Structure on Environment.

(a) Physical. Scour has taken place under the northwest corner of the pontoon base to a depth of 10 feet. It is to be remedied with triangular concrete blocks surrounding the area to deflect the current, with the space under the pontoon and in the immediate vicinity to be filled with grout, both in plastic bags and pumped into place.

Raw sewage is now being discharged at sea bottom level, but a sewage treatment plant is under construction.

(b) Biota. Because of the heavily silted water, this area has never been a good fishing ground. Fish are in the area during salmon runs, apparently unaffected by erection of the drilling structures. Waterfowl in the area are plentiful.

(c) Aesthetics. Aesthetics have had little influence on the design of the structures.

(9) Engineering. Brown and Root, Incorporated, Houston, Texas 77002.

(10) Construction Contractors.

(a) Onshore Prefabrication. American Pipe and Construction Company, Portland, Oregon 97208 (west coast fabricator chosen to eliminate tow through Panama Canal).

(b) Offshore Erection. Brown and Root Marine Operators, Houston, Texas 77022.

(11) Construction Dates.

(a) Onshore. October 1965 to May 1966.

(b) Offshore. June 1966 to October 1966.

(12) Construction Cost (approximate).

(a) To setting of substructure,	\$ 8,000,000
(b) Installation of decks, and drilling equipment, etc.,	4,000,000
Total:	\$12,000,000.

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b. Platform B, Mississippi River, South Pass, Venice, Louisiana.

(1) Construction.

(a) Completed. 1968.

(b) Type. Pile-supported, template-type structure (Fig. 60). Steel main piles and skirt piles driven through steel-trussed template. Superstructure of prefabricated sections.

(c) Purpose. Support of offshore oil drilling and production facilities.

(2) Owner. Shell Oil Company, New Orleans, Louisiana 70013.

(3) Location (approximate). South Pass, Block 70, off Mississippi River Delta, southeast of Venice, Louisiana.

Latitude: 29°00′N. – Longitude: 88°55′W.

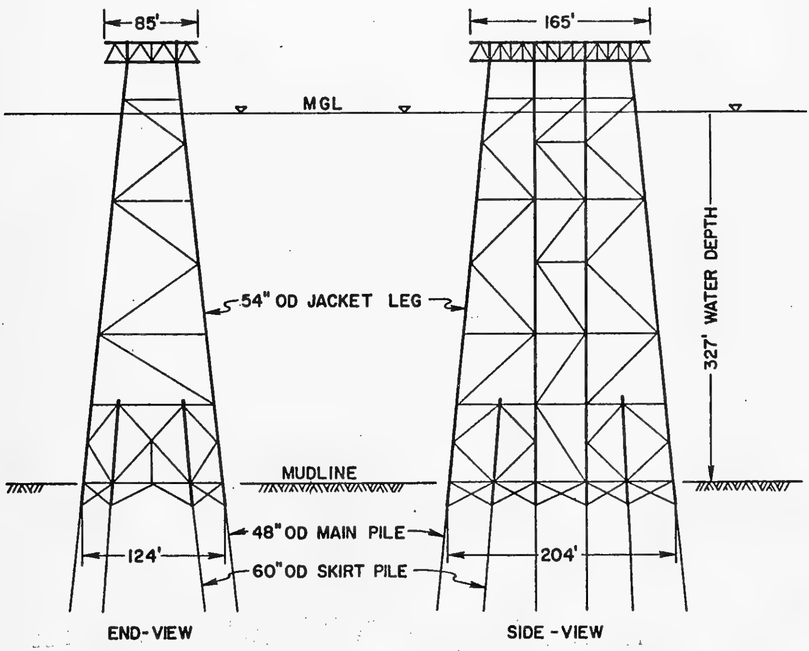

Figure 60. Structure as installed in 1968 (Sterling and Strohbeck, 1973).

(4) Physical Environment (Fig. 61)

(a) Open Estuarine Environment. Some protection to northwest from Mississippi River Delta land; full exposure in all other directions to wind and wave conditions of Gulf of Mexico.

(b) Wave Conditions. 72-foot wave measured on wave staff at a point 5 miles from Platform B location.

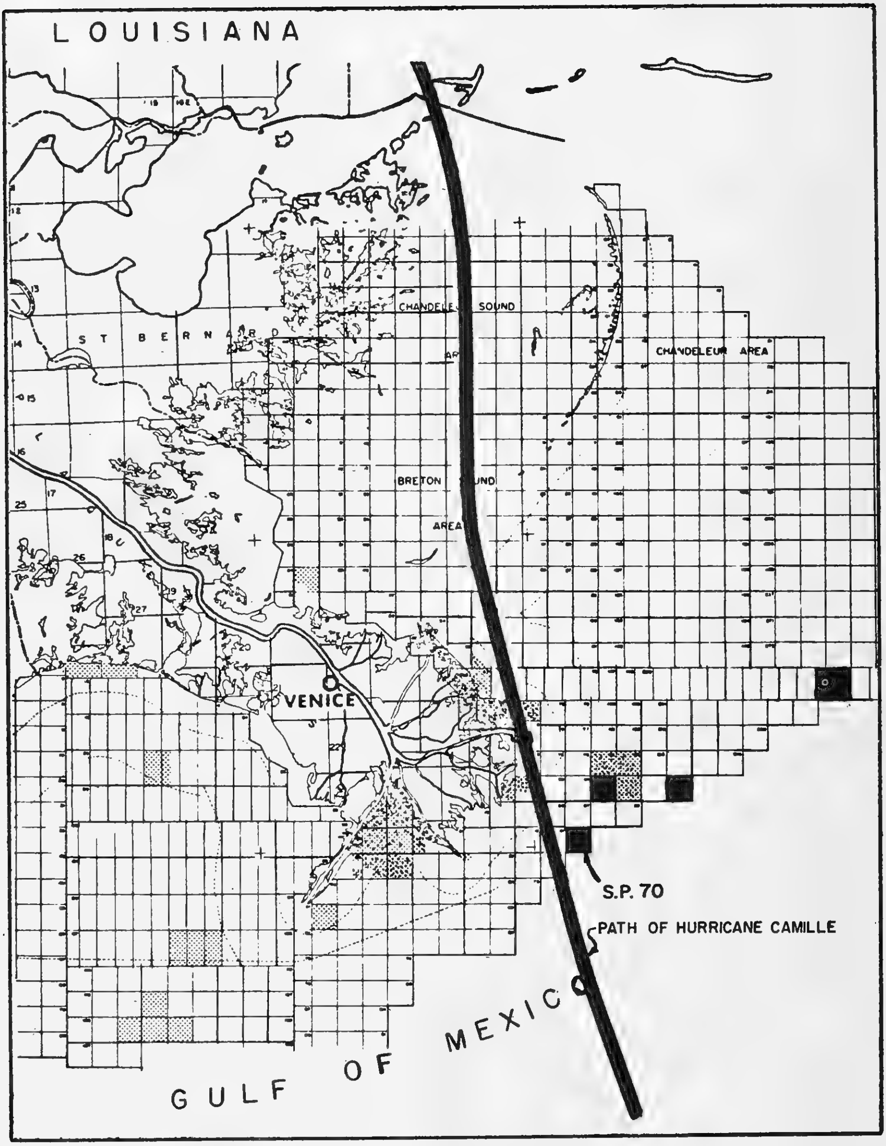

Figure 61. Location plan (Sterling and Strohbeck, 1973).

(c) Currents. Not significant.

(d) Winds. Up to 150 miles per hour during hurricane season, usually in late summer and fall.

(e) Storm Surge and Tide. Normal tidal range, 4 feet (approximate); storm tide, 10 to 12 feet.

(f) Littoral Transport. None, due to offshore location.

(g) Water Depth at Structure. Bottom at 327 feet below mean water level.

(h) Bottom Conditions. Soft clay soil.

(5) Structural Features (Fig. 60). Drilling and production platform of steel construction, 85 by 165 feet was supported on 8- to 48-inch-diameter main piles and 8- to 60-inch-diameter skirt piles, driven by means of template placed on bottom.

(6) Design Data. The design wave as recommended by oceanographer consultants was that forecast for a 100-year storm. Due to the owner's knowledge of the area, an extensive soil investigation program was carried out, with the 100-year criteria used for foundation design in connection with sea floor soil movements.

The high cost of a platform installed in over 300-foot depths requires a design close to minimum specifications; for a platform in shallower water, a more conservative design is still economically feasible.

(7) Structural Performance. In August 1969, Shell Oil Comany's Platform B was lost during Hurricane Camille (Fig. 62). As the result of an extensive investigation, data have been collected showing conclusively that the primary cause of failure was movement of the sea floor.

Although the existence of weak clay soils in the area was known, the weakness and the depth to which it extended were both underestimated. It is felt now that the exceptionally high wave which was experienced induced soil movements fatal to the foundation piling.

During the hurricane, 72-foot waves were recorded on a wave staff within 5 miles of the platform. Although this may not have been excessive for the 100-year storm, the effect on the sea bottom was more than anticipated. Frequency estimates for such combined conditions in the area vary from once in 75 years to once in 400 years, with current estimates being once in 250 years. Post-hurricane studies indicate that forces exerted on the structure by water alone were only 60 to 70 percent of the design force.

The above opinion is strengthened by the results of soil samples taken within weeks after the storm and a year later, and comparing these with the samples taken prior to installation. The samples show a drastic change in bottom level and soil strengths taking place during the storm, and then remaining without appreciable change for the following year.

Topographic surveys and side-scan sonar runs, in addition to the soil borings, show as much as 8 feet of subsidence in the immediate area of the structure and a 35-foot buildup in nearby areas, all of which falls into a general pattern of down-slope movement throughout Block 70 (Sterling and Strohbeck, 1973).

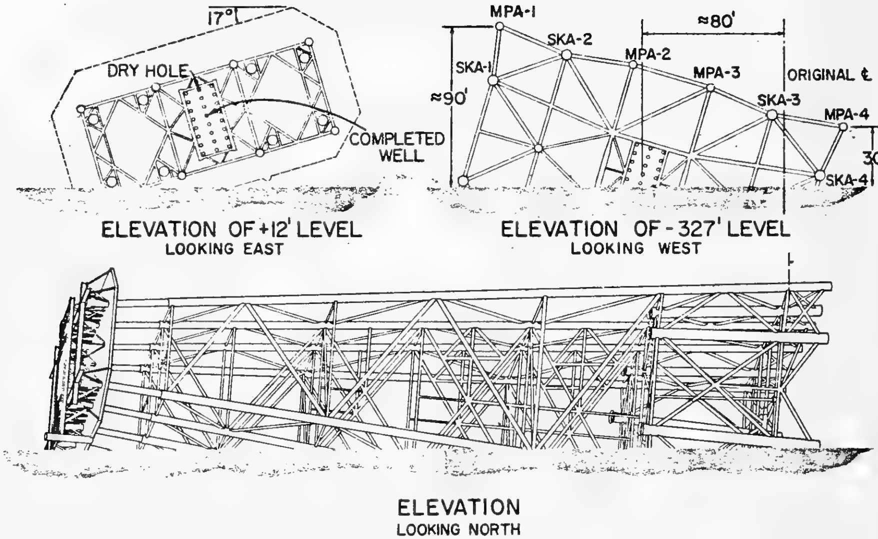

Figure 62. Structure after Hurricane Camille in August 1969 (Sterling and Strohbeck, 1973).

The owners have installed other platforms in similar water depths since the failure of Platform B, although not in areas of similar soil conditions. To date (1973) these platforms are performing successfully. Platform B is being replaced less than 0.25-mile to the northwest of the original site, but in an area of hard sand and calcareous material. At this point it will be left to the drillers' forces to extend their operations to reach the original source of oil.

It is of interest to note that pollution from broken oil lines due to storm damage (with several hours' warning in which to shut down) is far less of a problem than fire or collision (no warning and no time to shut down).

(8) Effect of Structure on Environment. Not relevant.

(9) Engineering. Not available.

(10) Construction Contractors. Not available.

(11) Construction Date. Not available.

(12) Construction Cost. Not available.

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